# OVERSTRENGTH AND DUCTILITY CAPACITY OF REINFORCED CONCRETE STRUCTURAL WALLS

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in Partial Fulfillment of the Requirements
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**MASTER OF TECHNOLOGY**

*by*
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**to the**
**Department of Civil Engineering**
**Indian Institute of Technology Kanpur**

# CERTIFICATE

It is certified that the work contained in this thesis entitled *"Overstrength and Ductility Capacity of Reinforced Concrete Structural Walls"* by Mr. Ranjith Kumar Reddy Jogala, has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

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# ABSTRACT

Shear wall systems have shown excellent performance in the past earthquakes. Lateral load response of reinforced concrete shear walls has been studied here. A computer program has been developed to obtain moment-curvature relationship for walls considering the boundary elements with confined concrete. This has been used to develop the lateral force – roof displacement relationship for building systems with shear walls.

An example building system, with six- and nine-storeys, has been used to study the behaviour. It is assumed to be located in seismic zone IV or V of Indian seismic zone map. The walls are designed in two ways: with boundary elements having special confining reinforcement as per IS: 13920-1993 and with boundary elements having only nominal shear reinforcement as per IS: 456-1978. Response is studied in terms of ductility, drift capacity, overstrength and response reduction factor.

Initial stiffness, yield curvature, and overstrength are found to be insensitive to the confinement in boundary elements. Due to confinement in boundary element, drift capacity increased by 20% to 70% and the response reduction factor by 25% to 70%. Overstrength of the walls in seismic zone IV is found to be higher by 15% to 23% than that in seismic zone V. It is also noted that nine-storey buildings have higher overstrength (15% to 25%) than the six-storey buildings. No particular trend was observed in the response reduction factor with the variation in seismic zone and number of storeys.

Performance of the wall buildings is compared with the available results in the literature for a similar R.C. frame building. Wall systems clearly have lower drift capacity. Six-storey wall buildings are stiffer than the frame structures and have lower overstrength where as lateral stiffness and overstrength are quite comparable for the two systems in case of nine-storeys. These results are also extrapolated to study the expected response of a *dual system* consisting of walls and frames. Individual walls clearly had more lateral stiffness than the frames. The walls and frames yielded at about the same lateral displacement level in case of nine-storey building, while in the six-storey building the frames were well-within elastic limit when wall yielded. Limited results of this study indicate that such dual systems should be designed based on the non-linear push-over analysis.

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*Ranjith*

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# List of Symbols

| | |
|---|---|
| $C$ | Function of time-period of structure as in IS: 1893-1984 |
| DL | Dead load |
| EQ | Earthquake load |
| $I$ | Importance factor |
| $K$ | Performance factor |
| $L$ | Height of the wall |
| LL | Live load |
| $R$ | Response reduction factor |
| $C_{eu}$ | Maximum elastic base shear coefficient |
| $C_w$ | Code-prescribed unfactored design base-shear coefficient |
| $C_y$ | Maximum yield strength base shear coefficient |
| $f_{ck}$ | Characteristic cube strength of concrete |
| $f_y$ | Characteristic yield strength of reinforcing steel |
| $h_i$ | Height of the roof or floor at $i^{th}$ level |
| $l_p$ | Length of the plastic hinge |
| $l_w$ | Length of the wall |
| $M_y$ | Yield moment |
| $M'_y$ | First yield moment |
| $M_u$ | Ultimate flexural strength |
| $Q_i$ | Lateral force at the floor level i |
| $R_\mu$ | Ductility reduction factor |
| $V_b$ | Design base shear force |
| $w_i$ | Lumped weight at floor level i |
| $W$ | Total weight of the building |
| $\alpha_0$ | Basic horizontal seismic coefficient |
| $\beta$ | Soil foundation factor |
| $\delta_y$ | Yield displacement at roof |
| $\delta_p$ | Plastic displacement at roof |
| $\varepsilon_{cu}$ | Ultimate compressive stain in concrete |
| $\Delta$ | Roof displacement of the structure |
| $\Delta_{max}$ | Maximum roof displacement of the structure |
| $\Delta_y$ | Yield displacement of the structure |
| $\mu$ | Ductility |
| $\Omega$ | Overstrength |
| $\varphi_p$ | Plastic curvature |
| $\varphi_y$ | Yield curvature |
| $\varphi_u$ | Ultimate curvature |
| $\theta_y$ | Yield rotation |

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# Chapter 1

# Introduction

## 1.0 General

Structures located in seismically active areas have to withstand lateral forces generated due to earthquakes in addition to their primary purpose of carrying gravity loads. The performance of a structure during an earthquake depends on the intensity of the earthquake and the properties of the structure. However, the intensity of earthquake at a given site is a function of several factors, which are associated with a high degree of uncertainty. On the other hand, sufficient level of reliability can be kept on quality of structure, as it depends on the well defined structural properties like configuration of the structural system, the analysis and design procedure, the detailing of the structural elements and skilful construction.

Adequate stress transfer paths must be provided in the structure in order to ensure that the inertia forces generated in the structure can be transmitted down to the foundation. This can be achieved by introducing a properly designed and detailed lateral force resisting system in the structure.

## 1.1 Lateral Force Resisting Structural Systems

The fundamental lateral force resisting systems are moment resisting frames, structural wall systems, braced frames or a combination of these basic structural systems (Figure 1.1). The selection of structural systems for buildings is influenced by the intended function, architectural considerations, internal traffic flow, height and aspect ratio and to lesser extent by the intensity of loading (Paulay and Priestley, 1992).

## 1.2 Structural Walls

*Structural walls* or *shear walls* (as these are popularly known) not only provide structural safety to buildings, but also provide a good protection against costly nonstructural damage by limiting the deflections. This has been clearly seen in Chile, 1985 earthquake (Wyllie et al., 1986). Structural walls increase stiffness of buildings significantly and increase the safety against collapse by suppressing the second-order effects. Favourably positioned structural walls, when properly designed and detailed for strength and ductility, are particularly effective during severe earthquake ground motion, where a considerable amount of energy input is to be dissipated by reversed inelastic deformations. Even after extensive cracking structural walls retain most of their vertical load carrying capacity, which is not always the case with columns.

Structural walls are usually conceived as vertical plates supported at foundation and are expected to function under the action of gravity and in-plane horizontal loads. However, based on the structural and architectural requirements of the building, shape of the structural walls is selected. Typical of them are rectangular sections, walls with flanges, walls which are cast in between two columns forming I-shaped section (barbell or dumbbell shape), and box sections which are often located as central core of a building.

### 1.2.1 Classification of Walls

Considering the behaviour, walls can be classified as slender shear walls and squat shear walls. Height-to-width ratio (or aspect ratio, *h/w*) is used to classify them. Shear walls with aspect ratio greater than 2.0 are called slender shear walls and with aspect ratio less than 0.5 are called squat shear walls (Fintel, 1974). The behaviour of walls is controlled by either shear or flexure, depending on the moment-to-shear span ratio ($M/VL_w$) at each horizontal cross-section of the wall. Flexural behaviour generally

predominates in slender walls and shear is likely to govern the behaviour of squat walls (Salse and Fintel, 1973).

### 1.2.2 Behaviour of Slender Structural Walls

The failure pattern in slender shear walls can be broadly classified as flexural failure, shear failure, sliding failure, and failure due to rotation of foundation (Figure 1.2). Of these, flexural failure due to yielding of longitudinal reinforcement at the edges of the wall [Figure 1.2 (a)] is the preferred source of energy dissipation due to its stable energy dissipating characteristics. The undesirable failure modes are diagonal tension or diagonal compression caused by shear before full flexural strength of the wall is developed [Figure 1.2 (b)], the sliding shear along the construction joints [Figure 1.2 (c)], and the failure of wall due to rotation of foundation [Figure 1.2 (d)].

### 1.2.3 Behaviour of Squat Structural Walls

The possible failure modes of a squat structural walls are schematically shown in Figure 1.3 (Paulay et al., 1982). When horizontal shear reinforcement is insufficient to resist the shear forces generated at the floor level, a corner-to-corner diagonal tension failure plane may develop [Figure 1.3 (a)]. The diagonal tension crack may also develop along a steeper plane [Figure 1.3 (b)]. If the shear reinforcement provided is adequate, the wall may fail in diagonal compression by crushing of the concrete in the web due to the development of diagonal concrete compression strut [Figure 1.3 (c)]. Under the action of reverse cyclic loading, the compressive strength of concrete reduces considerably. Hence, the diagonal compression failure may occur at a much lower shear load, and the crushing of concrete may rapidly spread over entire length of the wall [Figure 1.3 (d)], which may ultimately lead to the failure of the wall. Therefore, the diagonal compression failure is highly undesirable in structural walls. The other prominent mode of failure in squat structural walls is sliding-shear failure [Figure 1.3 (e)], commonly associated with low

level of axial loading and high levels of shear stresses. The wall moves horizontally along the construction joint.

## 1.3 Philosophy of Seismic Design

The basic objective of the earthquake-resistant design of structures is, in the event of an earthquake the structure should possess an adequate reserve strength to enable it to preserve its functionality with limited damage. In addition, it should prevent loss of life due to collapse in the event of a very severe but infrequent earthquake shaking. The seismic design requirements can be summarised by defining the limit states of performance level of the structure:

*Serviceability Limit State*: During small and frequent earthquakes, all structural components should remain elastic without any damage.

*Damage Control Limit State*: For ground shakings of moderate intensity, the structures may undergo a little repairable structural and nonstructural damage.

*Collapse Limit State*: Structures should withstand the strongest earthquake expected during its lifetime without collapse and prevent loss of life. However, it may undergo structural and nonstructural damage, which may be irreparable.

Clearly, the elastic limit of the structures is allowed to be exceeded during earthquakes of moderate and higher intensity. This implies that the structures should be able to undergo inelastic deformations without significant loss of strength and stiffness. This structural property is defined as ductility. Hence, the basic design criterion requires providing sufficient strength to the structure and a corresponding ductility.

Modern codes rely on the above-discussed philosophy. These codes suggest a dynamic or static analysis in the elastic range for the structure under consideration, but seismic forces are reduced by a factor depending on the ductility level of the structure.

This factor is called response reduction factor ($R$). Response reduction factor varies with the lateral force resisting structural system and depends on the ductility and overstrength of the system, which are quantified by ductility reduction factor ($R_\mu$) and overstrength factor ($\Omega$). Response reduction factor can be written as

$$R = R_\mu \, \Omega \tag{1.1}$$

The global response of a structure can be plotted in terms of base shear versus roof displacement as shown in the Figure 1.4 (Jain and Navin, 1995), which can be idealised as linearly elastic-perfectly plastic. The elastic response of the structure is also shown in the figure. The ductility reduction factor and overstrength can be defined using this figure.

*Ductility reduction factor* reduces the elastic force demand to the yield strength of the structure. It is defined as the ratio of elastic force demand on the structure to the yield strength of the structure.

$$R_\mu = \frac{C_{eu} W}{C_y W} = \frac{C_{eu}}{C_y} \tag{1.2}$$

This factor reflects the energy dissipation capacity of the structure. It primarily depends on the ductility of the structure ($\mu$) defined as the ratio of maximum displacement to the yield displacement.

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{1.3}$$

The *overstrength factor* accounts for the inherently introduced overstrength in the structure designed according to code. It can be defined as the ratio of base shear force corresponding to the actual yield strength of the structure to the code prescribed unfactored design base shear force.

$$\Omega = \frac{C_y W}{C_w W} = \frac{C_y}{C_w} \tag{1.4}$$

The use of unfactored design shear force in the definition of overstrength is to enable the comparison of overstrength of the structures designed according to different codes, which use different load factors in design load combinations.

All possible factors that may contribute to the strength exceeding the nominal or ideal values are reflected in the overstrength of the structure. The main factors, which make the first significant yield in the structure much more than the design forces can be summarised as follows (also see Navin, 1993):

- Load factors applied to the code prescribed design seismic forces used in the design of structures.
- Gravity loads present on the structure during the actual seismic event are much less than factored gravity loads used in design.
- Strength reduction factors are applied on the material properties used in design.
- Larger member sizes and additional members provided from architectural or stability considerations. Larger member sizes provided to avoid non-uniform member sizes. Minimal requirements of member sizes and minimum percentages of reinforcement provided to meet the codal provisions.
- Higher strength of materials than the specified nominal values.
- Materials have higher strength under cyclic loads than under static loads.
- Special seismic detailing requirements also contribute to additional strength.
- Non-structural elements and structural non-seismic elements contribute to the lateral strength, whose contribution is not accounted in the design.

The capacity of structures to take load even beyond the significant first yield in the structure, i.e., in the inelastic range is due to the following:

- Redundancy in the structure, which help in the redistribution of the internal forces.
- Additional strength enhancement of steel due to strain hardening at large deformations.
- Confinement of concrete helps in increase of ultimate moment capacity of RC sections.

Based on the previous experimental and analytical studies, it has been well established that the reserve strength of the structure in the form of overstrength has a significant role to play in the survival of buildings subjected to severe ground shaking. Blume (1977) has discussed a number of factors that contribute to the overstrength of the structure. It has also been noticed that overstrength is inherently introduced in buildings deigned according to the code (Bertero et al., 1991). Dynamic analysis results by Cassis and Bonelli (1992) on walls, frames and dual wall-frame systems indicated an overstrength value of 3-5. It was noted that wall systems generally have a higher value of overstrength than the frame systems. This study also made apparent the importance of higher wall area to floor plan area ratio in limiting the nonstructural damage. Studies conducted by Meli (1992) on two buildings for different ground motions revealed the strong dependence of the overstrength on the characteristics of ground motion and the redundancy of the structure. Zhu et al. (1992) noticed the fact that low-rise buildings have higher overstrength than high-rise buildings. Also, they demonstrated that buildings designed for a lower seismic force, with the same gravity loads generally exhibit higher values of overstrength. They observed the values of overstrength of a four-storey reinforced concrete ductile moment resisting frame in three seismic regions of Canada as 1.71, 1.43, and 1.23 (the higher value corresponding to the zone less prone to earthquakes). Non-linear pseudo-static analysis results by Jain and Navin (1995) on multi-storey reinforced concrete frames designed for seismic zones I to V as per Indian

code, showed strong dependence of overstrength on seismic zone. The average overstrength of frames in zone V and zone I was found to be 2.84 and 12.7, respectively. Moreover, it has been pointed out that overstrength increases with decrease in number of stories. Overstrength of 3-storey frame is higher than that in the 9-storey frame by 36% in zone V, and 49% in zone I. It has also been observed that the interior frames will have higher overstrength (17 to 47% in zone V and zone I, respectively) compared to the exterior frames.

## 1.4 Scope of the Study

A computer program has been developed to determine the moment-curvature relationship of reinforced concrete structural walls with confined boundary elements. So developed moment-curvature relationship is used as a tool to determine the behaviour of an example building system with structural walls as lateral force resisting system. The structural walls of the building are designed and detailed as per Indian codes. Further, lateral load response of the system is studied in terms of ductility capacity, drift ratio, and overstrength of the building using the global response of the structure (base shear versus roof displacement). Variation of these quantities in different seismic zones and for different number of storeys is also studied. The response of the shear wall buildings is compared with that of the moment frame buildings with similar plan studied by Jain and Navin (1995).

## 1.5 Organisation

The thesis has been divided into five chapters. Chapter 1 is an introduction. Chapter 2 gives the analysis procedure employed to determine the moment-curvature relationship of structural wall section with confined boundary elements. The idealised

definitions of yield curvature and ultimate curvature of walls used in the present study are also discussed. Chapter 3 gives the description of the example building and the design procedure used for structural walls. The analysis procedure used to relate the local deformations with the global deformations of the structural wall are also discussed. Chapter 4 presents results and discussions of the study conducted on the example building. Also, the results are compared with those for the similar moment frame building. Chapter 5 contains a brief summary and conclusions of the present study.

Appendix A gives the details of the stress-strain model used for confined and unconfined concrete in developing the moment-curvature relationship for structural walls. Appendix B provides detailed flow chart of the program developed and the program listing.

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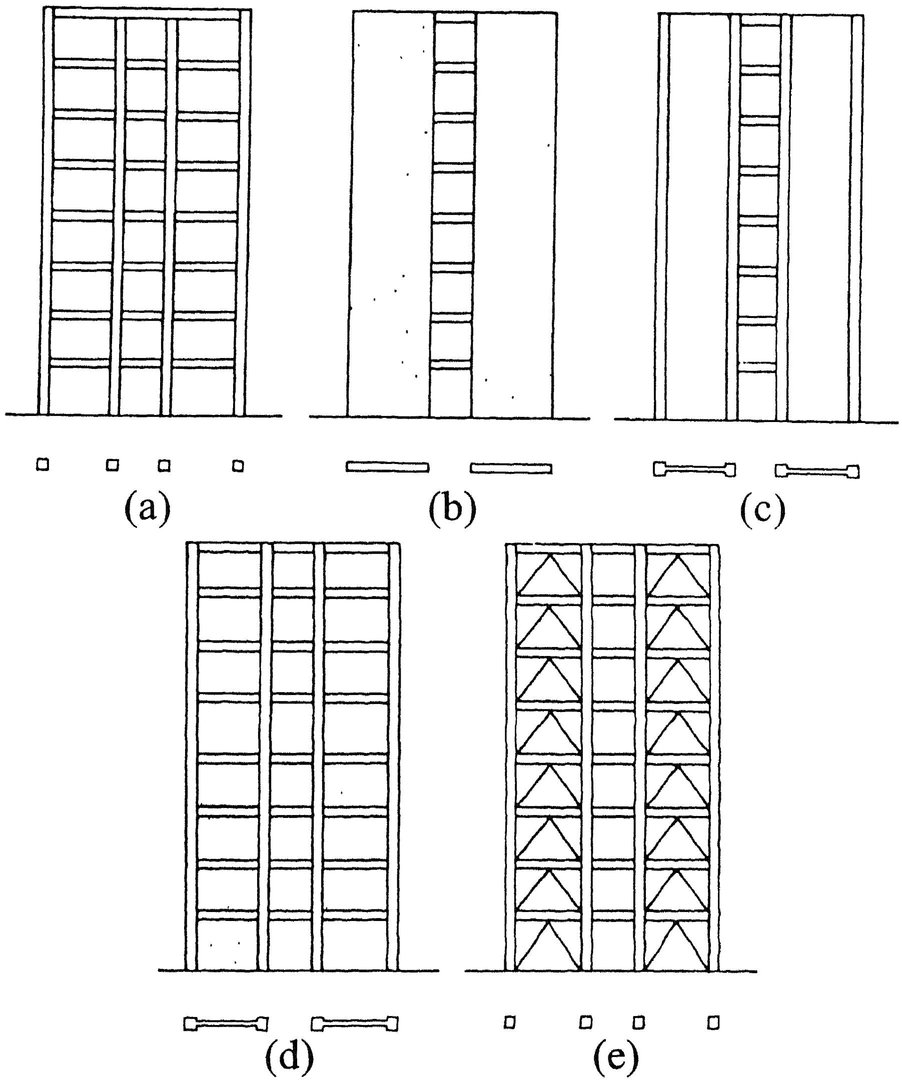

*Figure 1.1 Lateral-load–resisting systems (a) Pure frame (b) Shear wall (c) Shear wall with columns (d) Infilled shear wall (e) Braced frame (from Wakabayashi, M., 1986).*

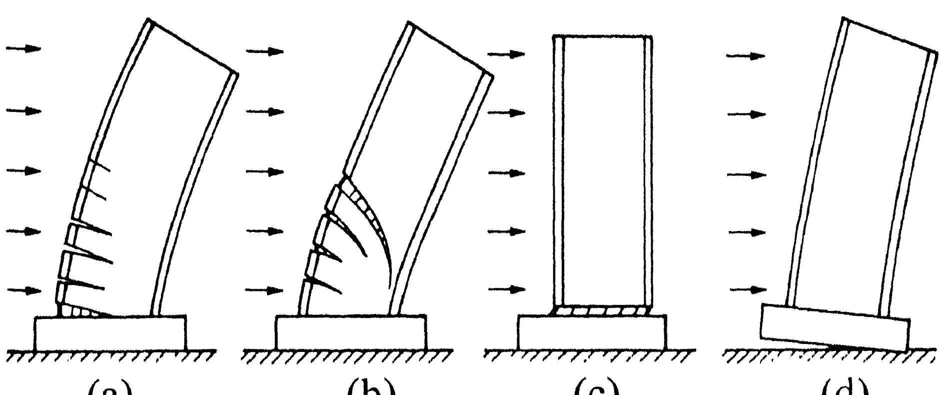

*Figure 1.2 Failure modes of slender structural wall (a) Flexural failure (b) Shear failure (c) Sliding failure (d) Rotation of foundation (from Wakabayashi, M., 1986).*

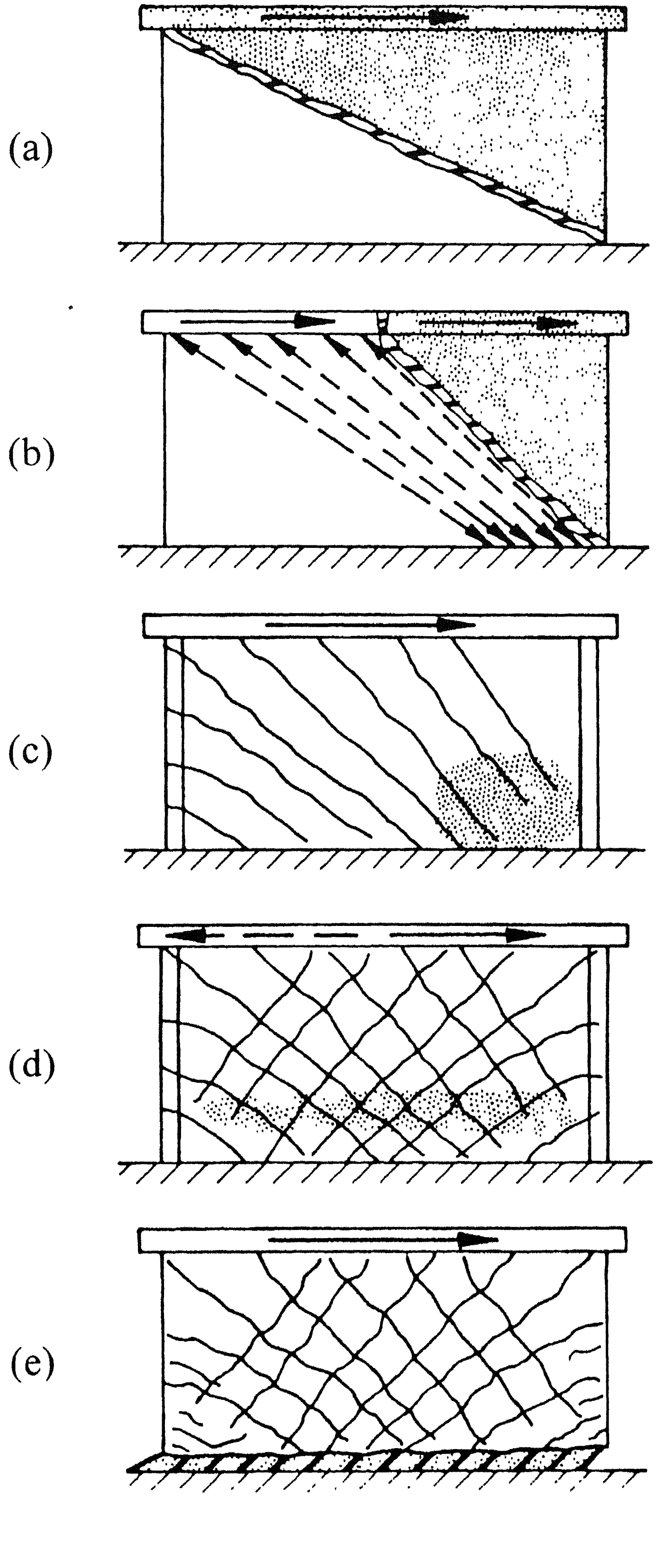

*Figure 1.3 Failure modes in squat structural walls (a) Diagonal tension failure (b) Diagonal tension crack along a steeper plane (c) Diagonal compression failure (d) Spread of crushing of concrete over the entire length of wall (e) Sliding-shear failure (from Paulay et al., 1982).*

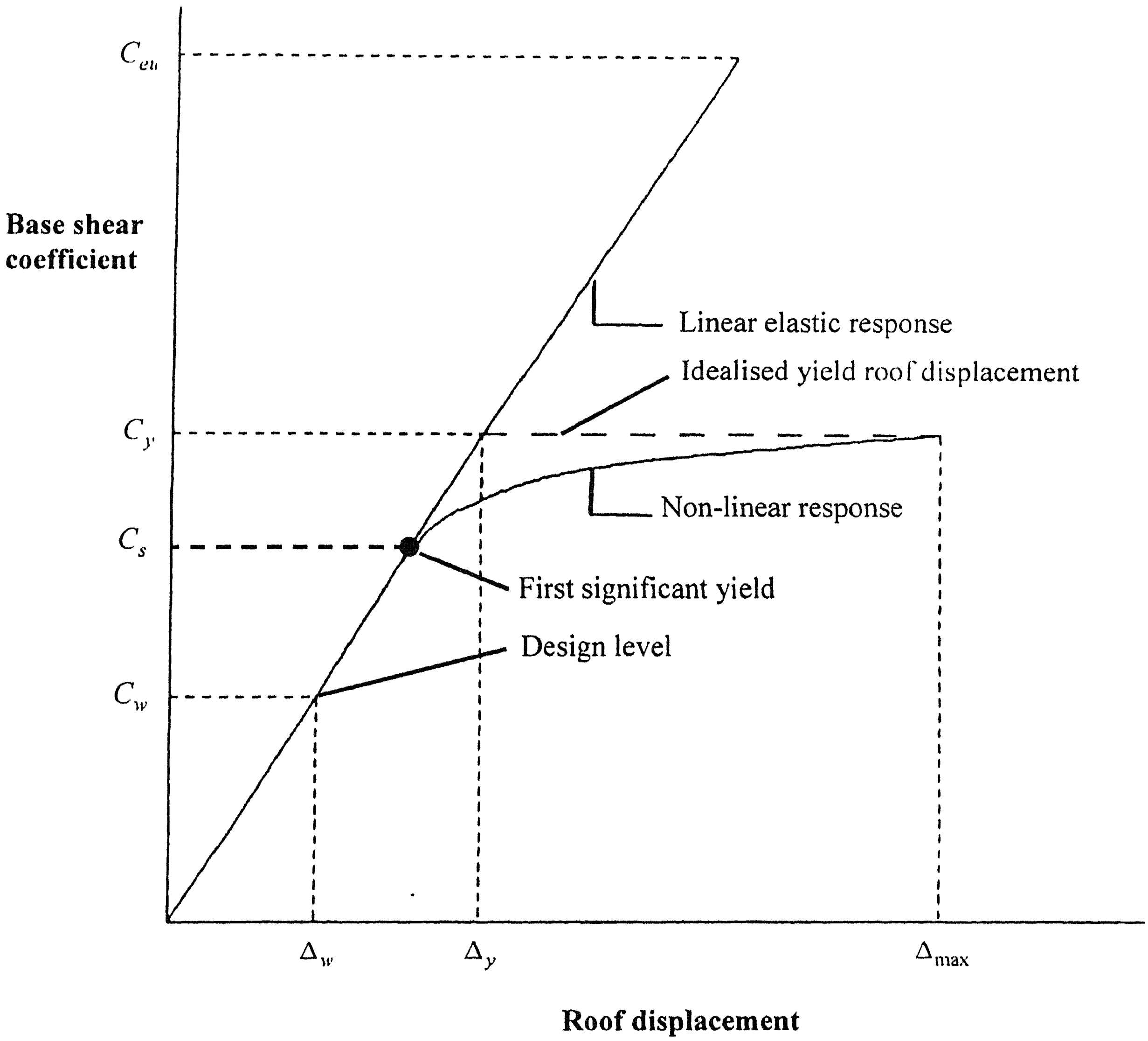


***Figure 1.4** Typical global structural response idealised as linearly elastic-perfectly plast curve (from Jain and Navin, 1995).*

# Chapter 2

# Moment-Curvature Analysis of Walls with Confined Boundary Elements

## 2.0 Introduction

This chapter presents the analysis procedure employed to determine the moment-curvature relationship for reinforced concrete structural walls with confined boundary elements under a given axial load. It also discusses the idealisation of moment–curvature relationship to define the yield curvature and the ultimate curvature of the walls. The following assumptions have been made in this study:

(a) Plane sections normal to the axis of bending remain plane after bending. This implies that the longitudinal strain in the concrete and steel at various points across a section is proportional to the distance from the neutral axis.

(b) Stress-strain relation for steel is known, and is identical in compression and tension.

(c) Stress-strain relation for concrete is defined, and the strength of concrete in tension is neglected.

The validity of the assumption (a) is found to be satisfactory from experimental studies conducted on squat shear walls with small aspect ratio ($h/w$) (Paulay et al., 1982). Based on experimental studies on shear walls under lateral loads, with and without the presence of vertical loads, Oesterle (1986) (also see Sasani, 1996) found that the calculated compressive strain assuming linear strain distribution is considerably less than the measured strains in the hinging region (base). He introduced the radially fanned cracked pattern as basis for developing compatibility relationship instead of the linear distribution of the strain along the section. However, the traditional assumption that plane

sections remain plane, used in developing a new methodology for the design of structural walls by Wallace (1994) was found to be consistent with an experimental study of a full-scale building. Hence, the same is used in the current study. The assumption (b) is reasonable when the transverse reinforcement provided is enough to avoid premature buckling of longitudinal reinforcement.

## 2.1 Stress-Strain Relations for Concrete and Steel Reinforcement

The stress-strain relations of concrete and reinforcing steel have significant effect on the moment-curvature relation of a reinforced concrete section.

### 2.1.1 Concrete

Significant amount of research has been done in understanding the behaviour of concrete, and a number of stress-strain models for both confined and unconfined concrete are available in the literature. Following an earlier review by Mandal (1993), in the present study also Saatcioglu and Razvi (1992) model (Figure 2.1) has been used for confined concrete. The generality of the model to cover majority of the concrete member shapes and the confinement schemes used in practice, and the relative ease in its use make it an appropriate analytical model for confined concrete. The stress-strain relationship established using this model were found to be in good agreement with those obtained from column tests of different geometry and reinforcement arrangement, conducted under concentric and eccentric loading. Appendix A gives the parameters required to define the stress-strain curve for confined concrete according to this model. The behaviour of unconfined concrete is modeled as per IS:456-1978 (Figure 2.2); this curve is also described in Appendix A.

### 2.1.2 Reinforcing Steel

The stress-strain relation for mild steel bars (Fe 250) and HYSD bars (Fe 415 and

Fe 500) have been taken from IS:456-1978 (Figure 2.3). The maximum elongation (fracture strain) of 23%, 14.5% and 12.0% for Fe 250, Fe 415 and Fe 500, respectively, are used as per IS:432 (Part 1) -1982, and IS:1786-1985.

## 2.2 General Methodology

Under the given axial load ($p$) and for an assumed cross-section strain profile it is possible to find the moment capacity ($M$) and the corresponding curvature ($\phi$) from the requirements of strain compatibility and the equilibrium of forces. Assuming that the applied axial load is uniformly distributed over the entire cross-section, uniform strain ($\varepsilon'_c$) in the section is determined. Starting with ($\varepsilon'_c$) as the extreme compression strain, curvature is varied to match the force equilibrium equation. The extreme compression strain and the internal forces so obtained are used to find the moment. Thus obtained moment and the corresponding curvature give one point on the moment-curvature plot. To complete the curve, the extreme compression strain is incremented in small intervals till the ultimate condition is reached. For every increment in strain the curvature is varied such that the force equilibrium is satisfied and the moment is calculated using corresponding strain and the internal forces. The following gives a more detailed view of the procedure.

The section is divided into n laminae of equal thickness ($H/n$), which are oriented along the neutral axis as shown in Figure 2.4(a). The concrete laminae, as well as the reinforcement bars are defined by the distance of their center line from the extreme compression (top) fiber. For a given extreme compression fiber strain ($\varepsilon_c$) and the curvature ($\phi$), the strain in the i^th^ lamina ($\varepsilon_i$), which is at a distance ($h_i$) from the top can be found from

$$\varepsilon_i = \varepsilon_c - \phi \ h_i \tag{2.1}$$

The strain within a concrete lamina is assumed to be constant as shown in Figure 2.4(b) and is found from the average of strains at the top and bottom of that lamina. The stress ( $f_{ci}$ ) in that layer is read from the corresponding stress-strain relation. The stress-strain relation for concrete is judged as confined or unconfined depending on the location of the lamina in the cross-section, i.e., whether the layer falls in the confined boundary region or in the unconfined region of cross-section. Force in concrete ($F_{ci}$) in the i[th] lamina can be computed from the stress in concrete and the concrete area in that lamina ($A_{ci}$).

$$F_{ci} = f_{ci} A_{ci} \tag{2.2}$$

Similarly, the strain in reinforcing bars in the j[th] lamina is found from the equation 2.1 by knowing the distance of the centroid of the bars from the extreme compression fiber. Hence, the stress ( $f_{sj}$ ) can be evaluated from the appropriate stress-strain curves for steel. Force due to steel ($F_{sj}$) can be computed from the stress in steel and the steel reinforcement area ($A_{sj}$).

$$F_{sj} = f_{sj} \ A_{sj} \tag{2.3}$$

The equilibrium of internal forces is checked with the applied axial load as follows

$$P = \sum_{i=1}^{n} F_{ci} + \sum_{j=1}^{m} F_{sj} \tag{2.4}$$

The corresponding moment ($M$) is then calculated by taking the moments of the internal forces about the centroidal axis.

$$M = \sum_{i=1}^{n} F_{ci} d_i + \sum_{j=1}^{m} F_{sj} d_j \tag{2.5}$$

Where, $n$ = number of laminae of concrete; $m$ = number of laminae of steel bars; $d_i$ = distance of center line of the $i^{th}$ lamina of concrete from the centroidal axis; and $d_j$ =

distance of center line of the $j^{th}$ lamina of steel bars from the centroidal axis. The contribution of all the vertical web reinforcement has been taken into account in the calculation of flexural strength.

## 2.3 Curvature Ductility

The ability of a structure to undergo large inelastic deformations without losing much of strength and stiffness is characterised by its ductility. Ductility is of paramount importance in earthquake-resistant structures as it enables one to design the structure for much lower forces than what is required for elastic behaviour of the structure.

The moment curvature curve of reinforced concrete walls is found to be strongly curvilinear due to the presence of multiple layers of reinforcement (Figure 2.5). Due to their curvilinear behaviour no specific yield point was observed on the curves. To define the curvature ductility of the wall, these curvilinear curves are transformed to bilinear ones. This transformation is done in different ways by different researchers. In the current study the following definitions are used:

*Ultimate Curvature* ($\phi_u$) is taken as the lower of the curvatures corresponding to

(a) The moment capacity of the section reducing to 80% of the maximum moment carried by the section.

(b) Extreme tensile reinforcing steel bar reaching fracture strain.

(c) The compression strain in the extreme fiber of core concrete in the confined boundary element attaining a value of $\varepsilon_{cu} = 0.004 + 0.9\rho_s\left(f_y/300\right)$, proposed by Scott et al. (1982).

Here, $\rho_s$ = ratio of volume of transverse reinforcement to the volume of concrete core; and $f_{yh}$ = yield strength of transverse reinforcement in (MPa).

*First yield* ($\phi'_y$) is taken as (Figure 2.6) the point ($\phi'_y, M'_y$) moment and curvature

corresponding to the strain in the extreme steel bar reaching $\varepsilon_s = f_y/E_s$ or the compressive strain at the junction of confined boundary element with the unconfined web reaching 0.002, whichever occurs first. *Yield curvature* (Figure 2.6) is assumed as the curvature found by extrapolating the line from the origin of the moment-curvature curve through first yield ($\phi'_y, M'_y$) to the ultimate flexural strength. Thus,

$$\phi_y = \phi'_y \, M_u / M'_y \tag{2.6}$$

Hence, the curvature ductility can be calculated as

$$\mu_\phi = \phi_u / \phi_y \tag{2.7}$$

## 2.4 Computations

Considering the analysis procedure given in this chapter, a computer program has been developed to compute the moment-curvature relationship of a reinforced concrete structural wall with confined boundary elements. This program is an improvement over an already available computer program (Mandal, 1993). The program has been modified accordingly, to take into account the confined and unconfined part of the cross-section. Appropriate stress-strain relationships have been used for concrete, in confined and unconfined part of the wall section. The program is explained in detail in Appendix B using a detailed flow chart and program listing.

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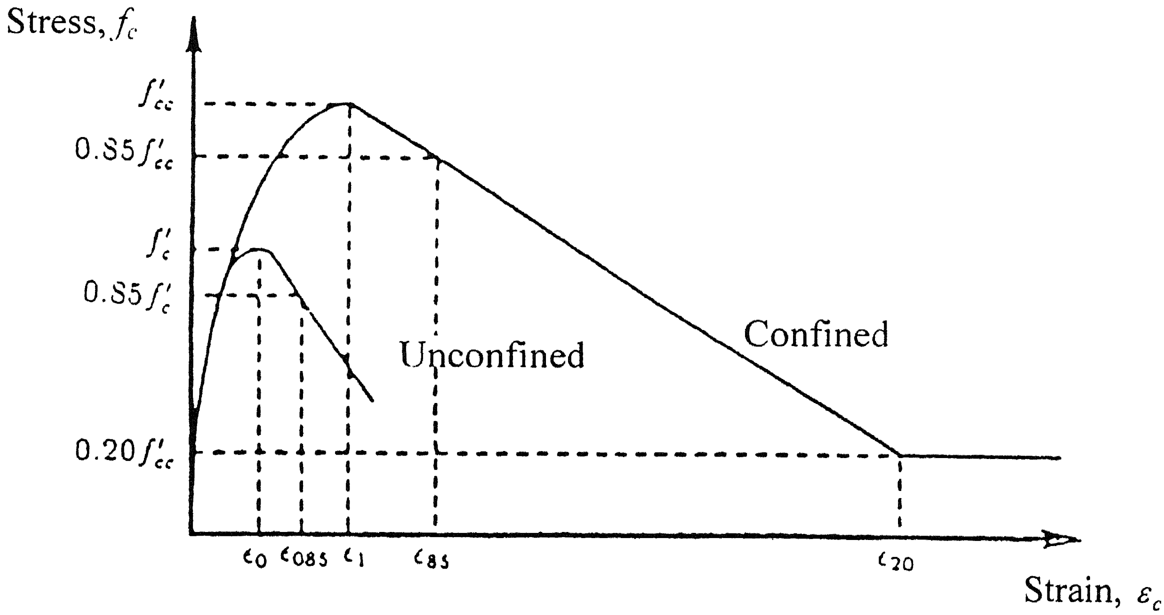



***Figure 2.1 Stress-strain relationship for concrete proposed by Saatcioglu and Razvi, (1992)***

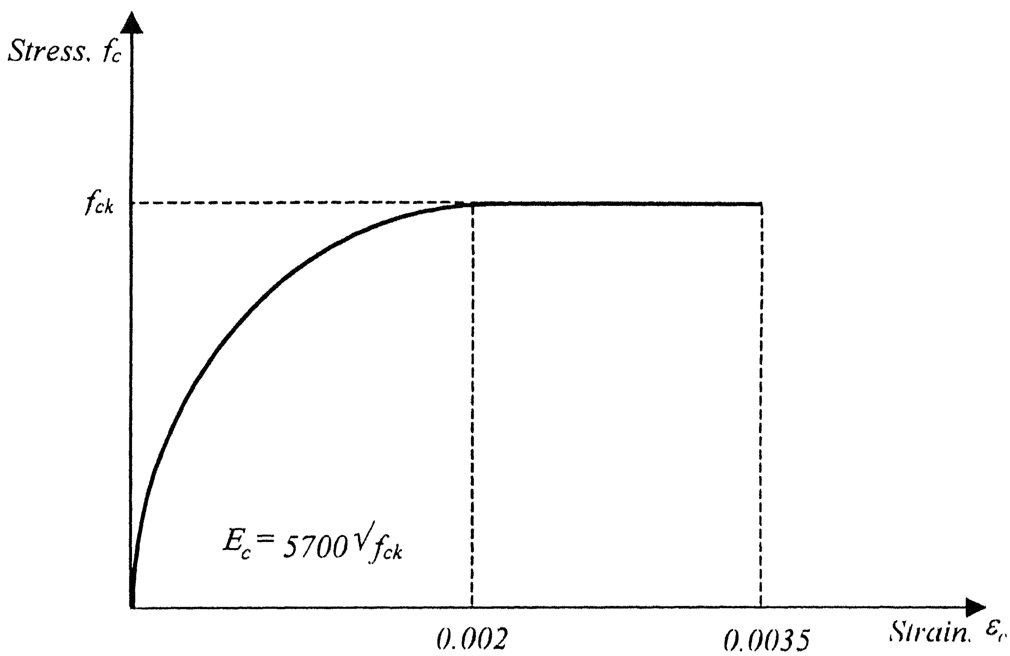
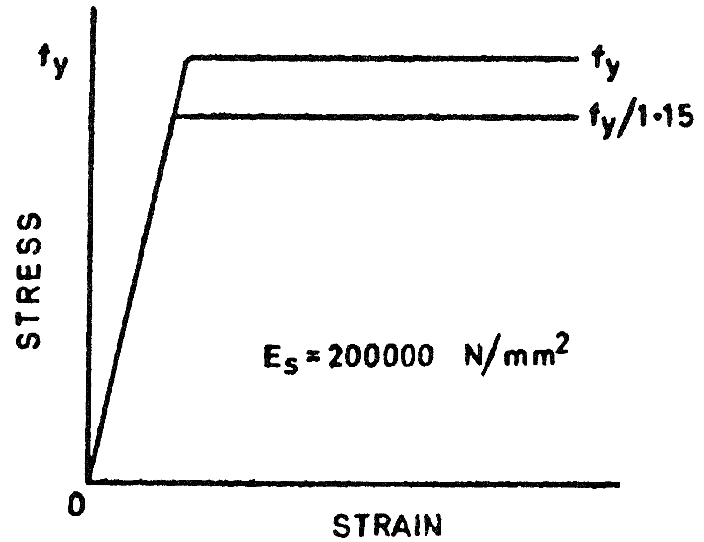
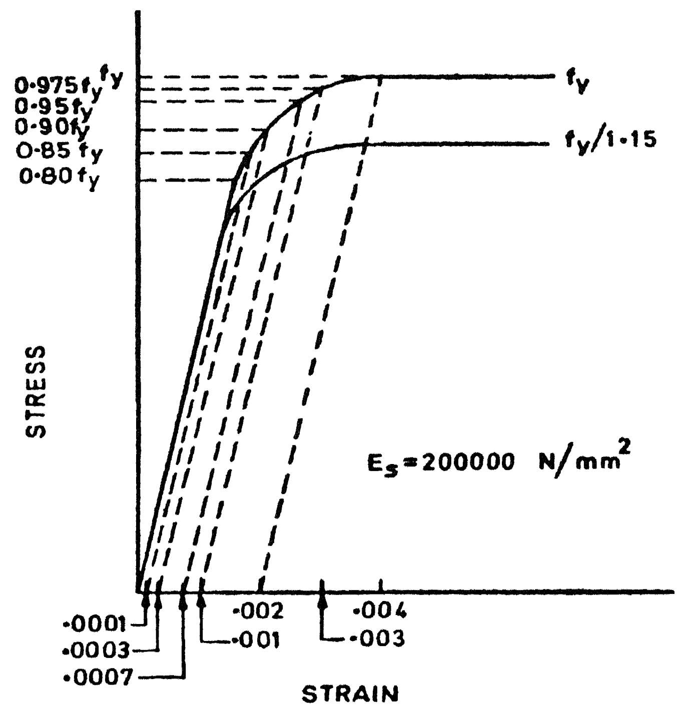



***Figure 2.2 Stress-strain curve for unconfined concrete used in the present study (from IS:456-1978)***




(a)




(b)

*Figure 2.3 Stress-strain of steel used in the moment-curvature analysis (a) for mild steel ($f_y$ = 250), and (b) HYSD bars (from IS:456 –1978)*

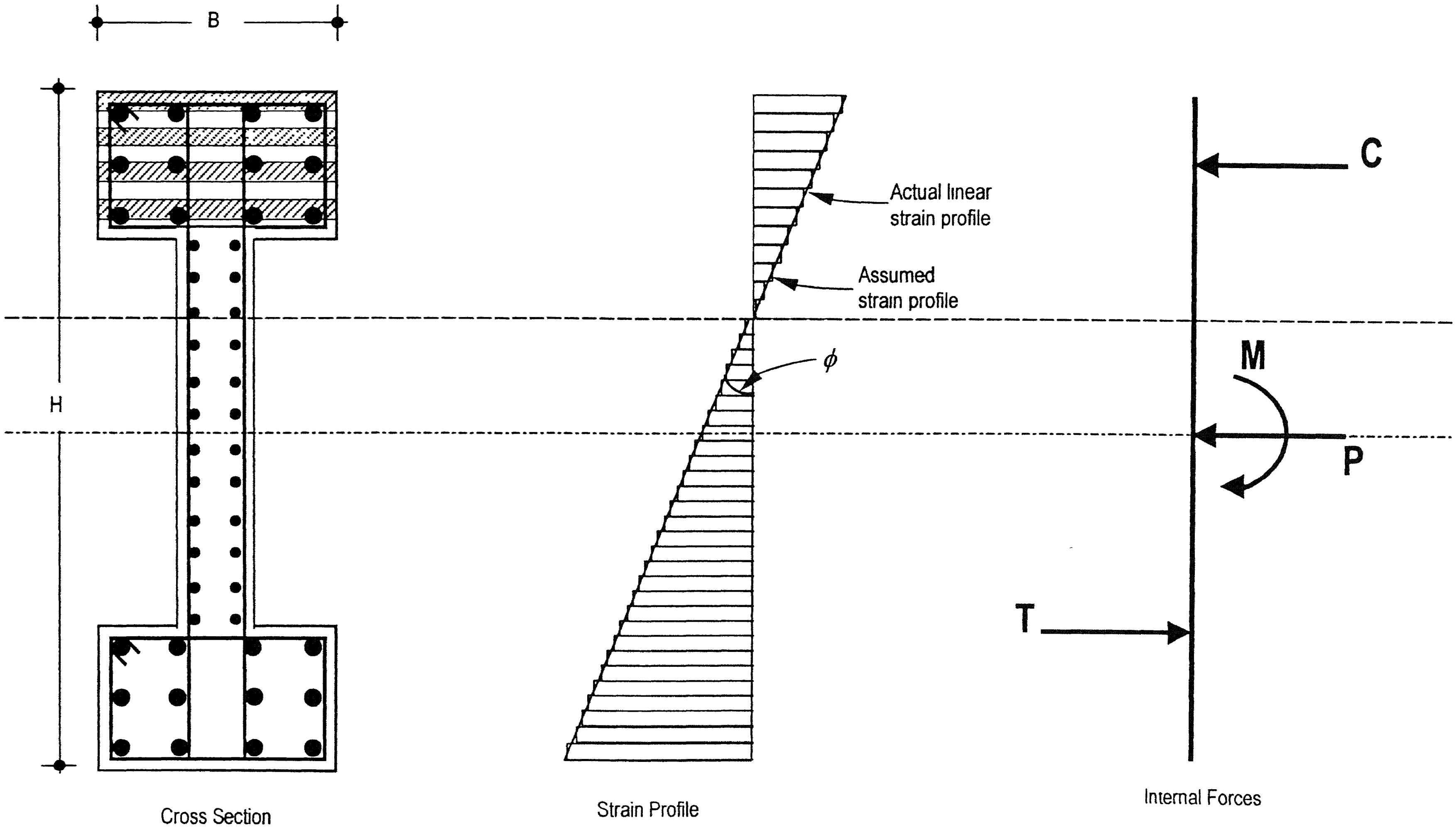



*Figure 2.4: Distribution of strain and internal forces across the cross section*

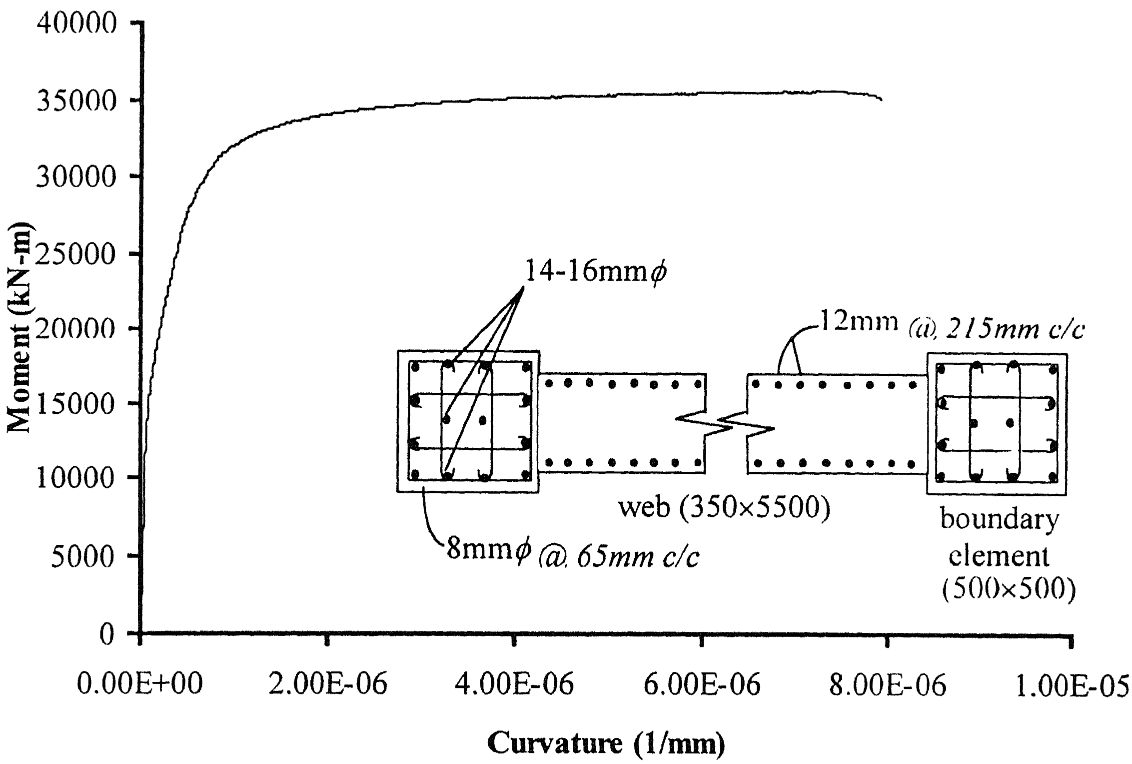



***Figure 2.5 Moment-curvature curve of structural wall SW65***

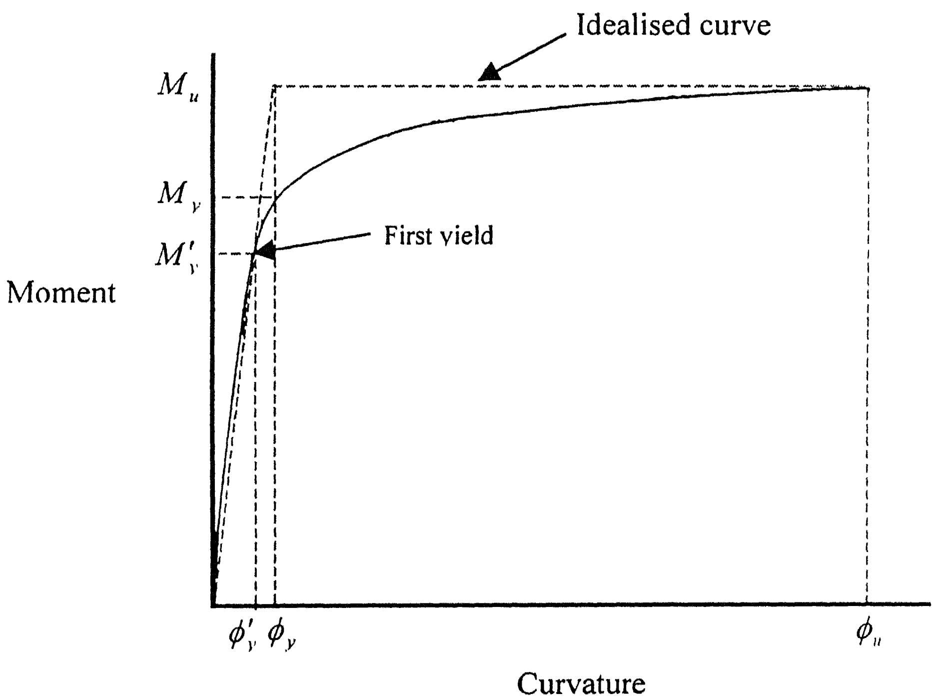



***Figure 2.6 Typical moment-curvature curve of a reinforced concrete structural wall***

# Chapter 3

# Analysis Procedure

## 3.0 Introduction

It is well established that overstrength inherently introduced in the code designed buildings plays major role in the survival of buildings subjected to severe earthquakes. Earlier, a comprehensive study was conducted to assess the performance of the moment resisting frames designed for seismic zones I to V as per Indian code (Jain and Navin, 1995). It was found that the overstrength of the structure strongly depends on the seismic zone. They have also pointed out the increase in overstrength values with decrease in number of storeys and the higher overstrength associated with the interior frames as compared to the exterior frames. It was considered useful to study the performance of the building with the same plan [Figure 3.1(a)] as in Jain and Navin, (1995) but with lateral force resisting system consisting of structural walls [Figure 3.1(b)]. This chapter presents the description of the building and the calculation of design lateral forces according to the code. Also given are the details of the structural walls designed and detailed as per Indian codes. The chapter also explains the analysis procedure used in this study to determine the performance of the code designed structural walls.

## 3.1 Building Description

The buildings used for the analysis are of six-storey and nine-storey located in seismic zones V, and IV. All the buildings are of same plan with 4 numbers of 6m bays in the longitudinal direction and 3 numbers of 6m bays in the transverse direction [Figure 3.1(b)]. Two structural walls are located in the longitudinal direction forming the lateral

force resisting system for the building in that direction. It is assumed that the two structural walls take the entire lateral loads coming on to the building in the longitudinal direction. Structural walls are positioned in plan such that the torsional effects are minimised in the event of severe shaking of building during an earthquake. The storey height is 4.4 m for the ground storey and 3.2 m for the rest of the storeys. The dead load is assumed as 10 kN/m$^2$ on the roof and 12 kN/m$^2$ on the floors. Live load was taken as 1.5 kN/m$^2$ on the roof and 4.0 kN/m$^2$ on the floors.

## 3.2 Design of Shear Walls

The design lateral force on the building was calculated as per IS: 1893-1984 using seismic coefficient method. The design base shear is given by:

$$V_B = K\, C\, \beta\, I\, \alpha_0\, W \tag{3.1}$$

Where, K is performance factor; C is a function of time period (T) of the structure; β is soil foundation factor (= 1.0); I is importance factor (= 1.0); $\alpha_0$ is basic horizontal seismic coefficient having a value of 0.08 and 0.05 for seismic zones V and IV, respectively; and W is the total seismic weight of the building. Here, the fundamental time period of the building is estimated using the formula T = 0.1n, where n is total number of storeys.

The design base shear force thus calculated is distributed over the height of the building based on the IS: 1893-1984 distribution formula, given by:

$$Q_i = V_B \frac{w_i\, h_i^2}{\sum_{i=1}^{n} w_i\, h_i^2} \tag{3.2}$$

where, $Q_i$ is the lateral force at the roof or floor level i, $w_i$ is the lumped weight (dead load + appropriate amount of live load, depending on the load class) at the floor level i, $h_i$

is the height of the roof or floor level i, measured from the base of the building. Table 3.1 gives the values of lumped weights ($w_i$) for six-storey and nine-storey buildings. The design lateral forces at different floor levels of the six- and nine-storey buildings located in seismic zones V and IV are given in Table 3.2. The design lateral force for each wall in the longitudinal direction is taken as half of the design lateral force for the complete building, keeping in view of their symmetrical location in the plan and assuming the rigid floor diaphragm action.

The structural walls are designed and detailed as per Indian codes using limit state method of design. Two types of detailing for the boundary elements are considered. In one, the boundary elements are provided closely-spaced shear stirrups as per IS: 13920-1993. In the other, the boundary elements have nominal shear stirrups as per IS: 456-1978. The nominal properties of the materials used in the design are given in Table 3.3. The following load combinations were used in the design as per IS: 456-1978:

(a) 1.5 (DL + LL)

(b) 1.2 (DL + LL $\pm$ EQ)

(c) 1.5 (DL $\pm$ EQ)

(d) 0.9 DL $\pm$ 1.5 EQ

Table 3.4 gives the geometric and the reinforcement details of the walls. Detailing in boundary elements as per IS: 456-1978 and IS: 13920-1993 is also given. The walls of six-storey building are designated as SW65 and SW64 in zone V and zone IV respectively. Similarly, the walls of nine-storey building are designated as SW95 and SW94 in zone V and zone IV respectively.

## 3.3 Deformation Capacity

The deformation capacity of walls can be evaluated using well-established

procedures (Park and Paulay, 1975) to account for the distribution of elastic and inelastic deformations over the wall height. The following procedure illustrates the determination of global deformations from the local deformations. The structural wall is assumed to be cantilever column subjected to lateral forces distributed over the height. Figure 3.2 shows the structural wall that has reached ultimate curvature and moment at the critical section i.e., at the base of the wall. The figure also shows the idealisation of the distribution of curvature along the height of the wall into elastic and inelastic regions. The elastic curvature distribution is idealised as a parabola with ordinate as the yield curvature ($\phi_y$). Once the wall attains a curvature of $\phi_y$, any additional deformation will be accommodated by inelastic rotation centered at the plastic hinge formed at the base of the wall. The inelastic deformation is spread over a height of the wall, this region being at least that, at which the bending moments are more than the yield moment of the wall. This plastic curvature is assumed to be constant over a height equal to the equivalent plastic hinge length $l_p$. The equivalent length of the plastic hinge can be estimated using empirical relations. In the present study the equivalent length of plastic hinge in walls is taken as (Paulay and Priestley, 1992):

$$l_p = 0.2 l_w + 0.044 L \tag{3.3}$$

where, $l_w$ is the length of the wall; and $L$ is the height of the wall. The plastic hinge length calculated from the equation (3.3) is not to be greater than 0.8 $l_w$ and not less than 0.3 $l_w$.

Using the idealised curvature diagram, the deformations of the wall can be determined. The rotation $\theta_y$ corresponds to the slope of the free end when yielding has occurred and is given by the area under the elastic curvature diagram. The elastic deflection of the wall can be obtained by taking the moment of the area of the curvature diagram about the tip of the wall. For the assumed parabolic variation of the curvature in the elastic region the elastic displacement of the wall is calculated as (refer to Figure 3.2):

$$\theta_y = \frac{3}{8}\phi_y L \qquad (3.4)$$

$$\delta_y = \left(\frac{3}{8}\phi_y L\right)\left(\frac{22}{30}L\right) = \frac{11}{40}\phi_y L^2 \qquad (3.5)$$

The plastic curvature capacity ($\phi_p$) of the section is given as the difference between the ultimate curvature ($\phi_u$) and the yield curvature ($\phi_y$) of the section. The plastic rotation $\theta_p$, is the area of the curvature diagram above $\phi_y$, shown shaded in the curvature diagram of Figure 3.2. This area is idealised as a parallelogram, whose moment about the tip of the wall gives the plastic deformation of the wall.

$$\phi_p = \phi_u - \phi_y \qquad (3.6)$$

$$\theta_p \cong l_p\left(\phi_u - \phi_y\right) \qquad (3.7)$$

$$\delta_p = l_p\left(\phi_u - \phi_y\right)\left(L - 0.5 l_p\right) \qquad (3.8)$$

Then, the total deformation of the tip of the wall can be calculated as sum of elastic and plastic deformations.

$$\Delta = \delta_y + \delta_p \qquad (3.9)$$

After the structural wall has been designed, its performance can be predicted using the above-discussed method. The deformation capacity of the structural walls can be evaluated by knowing the yield and ultimate curvature of the shear wall section. As explained in the previous chapter, the program developed to evaluate the moment-curvature relationship for the structural walls with confined boundary elements, gives the yield and ultimate curvature of the wall under the given axial load. The properties of the wall thus obtained at the cross-section level are used to study the behaviour of walls at global level.

• • •

***Table 3.1: Lumped weights at different levels of six-storey and nine-storey buildings***

| Level | Reactive weight (DL + appropriate amount of LL) in kN | |
|---|---|---|
| | Six-storey building | Nine-storey building |
| 9 | - | 4,320 |
| 8 | - | 6,048 |
| 7 | - | 6,048 |
| 6 | 4,320 | 6,048 |
| 5 | 6,048 | 6,048 |
| 4 | 6,048 | 6,048 |
| 3 | 6,048 | 6,048 |
| 2 | 6,048 | 6,048 |
| 1 | 6,048 | 6,048 |
| Total reactive weight of the structure (kN) | 34,560 | 52,704 |

***Table 3.2: Design lateral forces at different levels of six- and nine-storey building in seismic zones IV and V***

| Level | Design lateral forces (kN) | | | |
|---|---|---|---|---|
| | Six-storey building | | Nine-storey building | |
| | Zone V | Zone IV | Zone V | Zone IV |
| 9 | - | - | 502.4 | 314.0 |
| 8 | - | - | 561.4 | 350.9 |
| 7 | - | - | 435.4 | 272.1 |
| 6 | 627.0 | 391.8 | 325.6 | 203.3 |
| 5 | 622.0 | 388.8 | 231.1 | 144.5 |
| 4 | 413.5 | 258.4 | 153.2 | 95.7 |
| 3 | 245.9 | 153.6 | 91.1 | 56.9 |
| 2 | 121.7 | 76.0 | 45.1 | 28.2 |
| 1 | 40.9 | 25.5 | 15.1 | 9.4 |
| Total design Lateral force (kN) | 2,071.0 | 1,294.1 | 2,361.0 | 1,475.0 |

***Table 3.3 Properties of materials***

| | |
|---|---|
| Concrete:<br>Characteristic cube strength, $f_{ck} = 20$ MPa<br>Modulus of elasticity, $E_c = 25{,}500$ MPa | |
| Reinforcement:<br>Yield strength,<br>Modulus of elasticity, | <br>$f_y = 415$ MPa<br>$E_s = 20{,}000$ MPa |

*Table 3.4: Geometric and reinforcement details of the walls designed for six- and nine-storey building located in seismic zones V and IV*

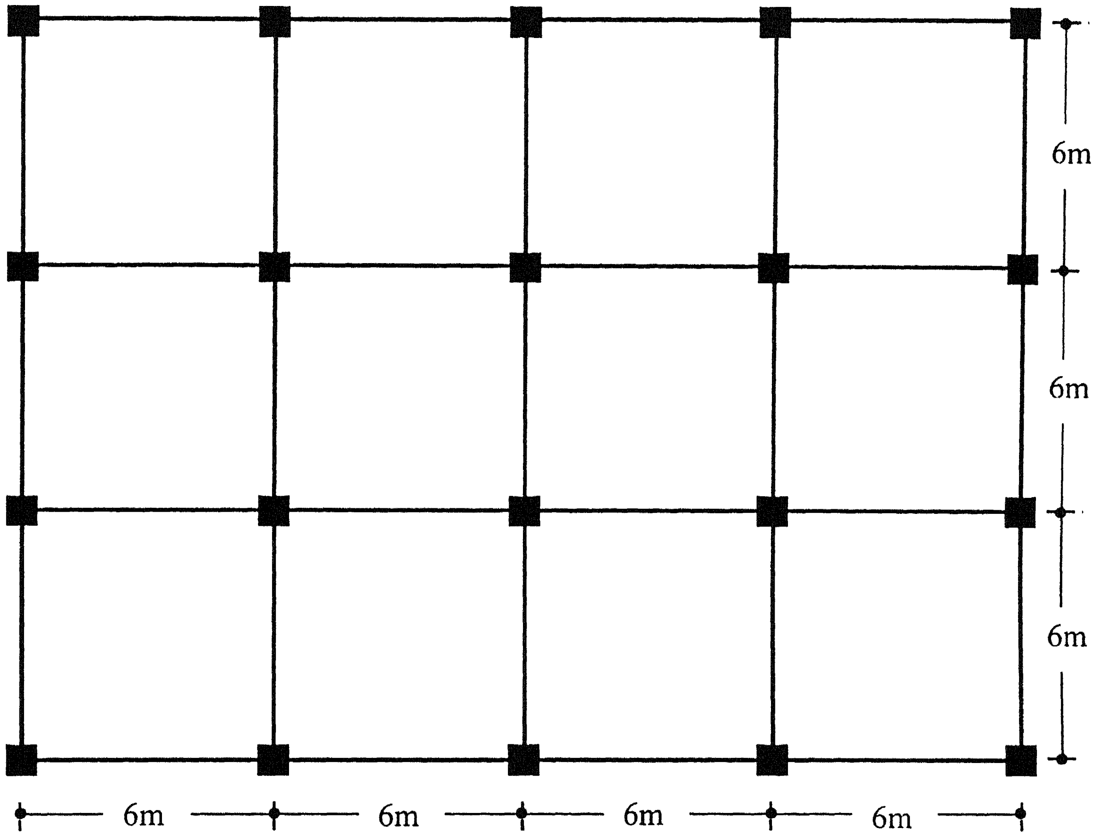
Geometric properties:

| Building | Wall | Thickness of web (mm) | Depth of web (mm) | Size of boundary element (mm×mm) |
|---|---|---|---|---|
| Six-storey building | SW65 | 350 | 5500 | 500×500 |
| | SW64 | 200 | 5600 | 400×400 |
| Nine-storey building | SW95 | 400 | 5300 | 700×700 |
| | SW94 | 150 | 5300 | 700×700 |

Reinforcement Details:

| Building | Wall | Web reinforcement (0.25%) | Longitudinal reinforcement in boundary element | Transverse reinforcement in boundary element | |
|---|---|---|---|---|---|
| | | | | IS: 13920-1993 | IS: 456-1978 |
| Six-storey building | SW65 | 52-Y12mm bars in two layers @ 215mm c/c | 14-Y16mm (1.12%) | Y8mm @ 65mm c/c | Y8mm @ 256mm c/c |
| | SW64 | 56-Y8mm bars in two layers @ 205mm c/c | 12-Y12mm (0.85%) | Y8mm @ 60mm c/c | Y8mm @ 192mm c/c |
| Nine-storey building | SW95 | 54-Y16mm bars in two layers @ 200mm c/c | 20-Y25mm (2.00%) | Y8mm @ 90mm c/c | Y8mm @ 384mm c/c |
| | SW94 | 64-Y8mm bars in two layers @ 160mm c/c | 24-Y20mm (1.54%) | Y8mm @ 90mm c/c | Y8mm @ 320mm c/c |

(a)




(b)

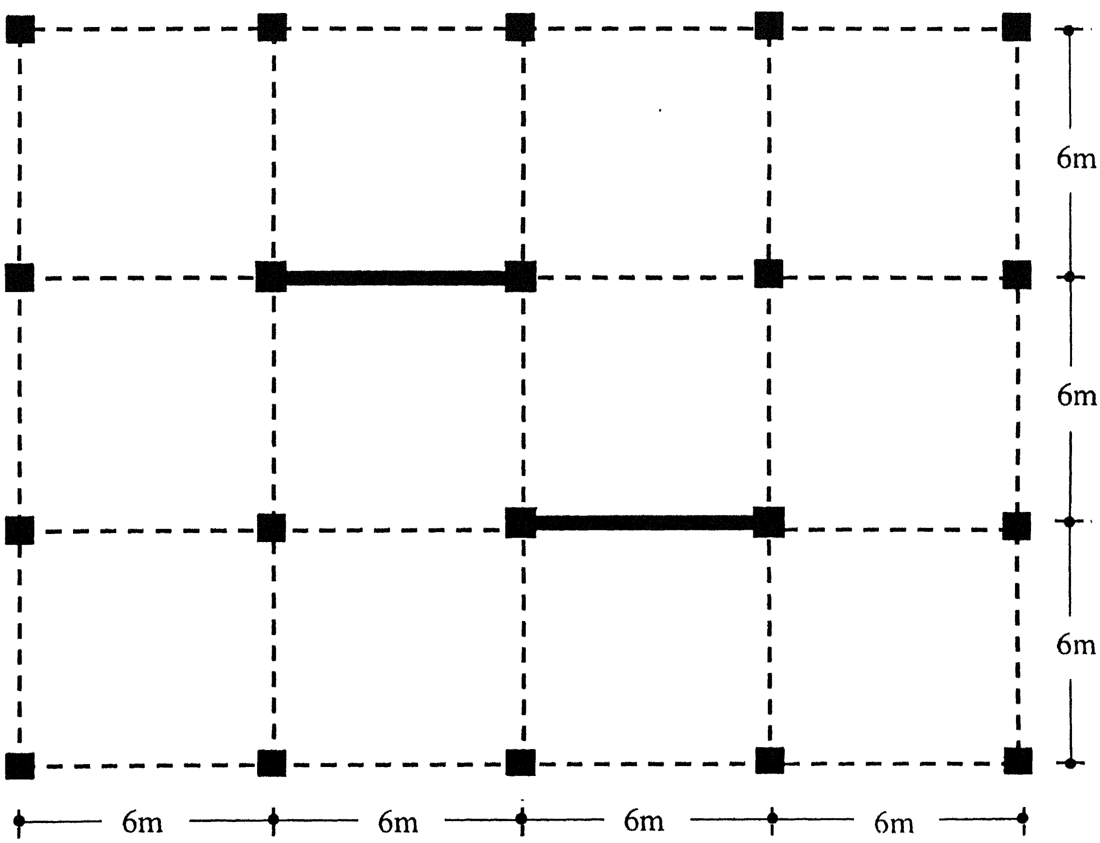



***Figure 3.1: Plan of the building (a) with frames as structural system (Jain and Navin, 1995) (b) with walls as lateral load system and columns for gravity loads***

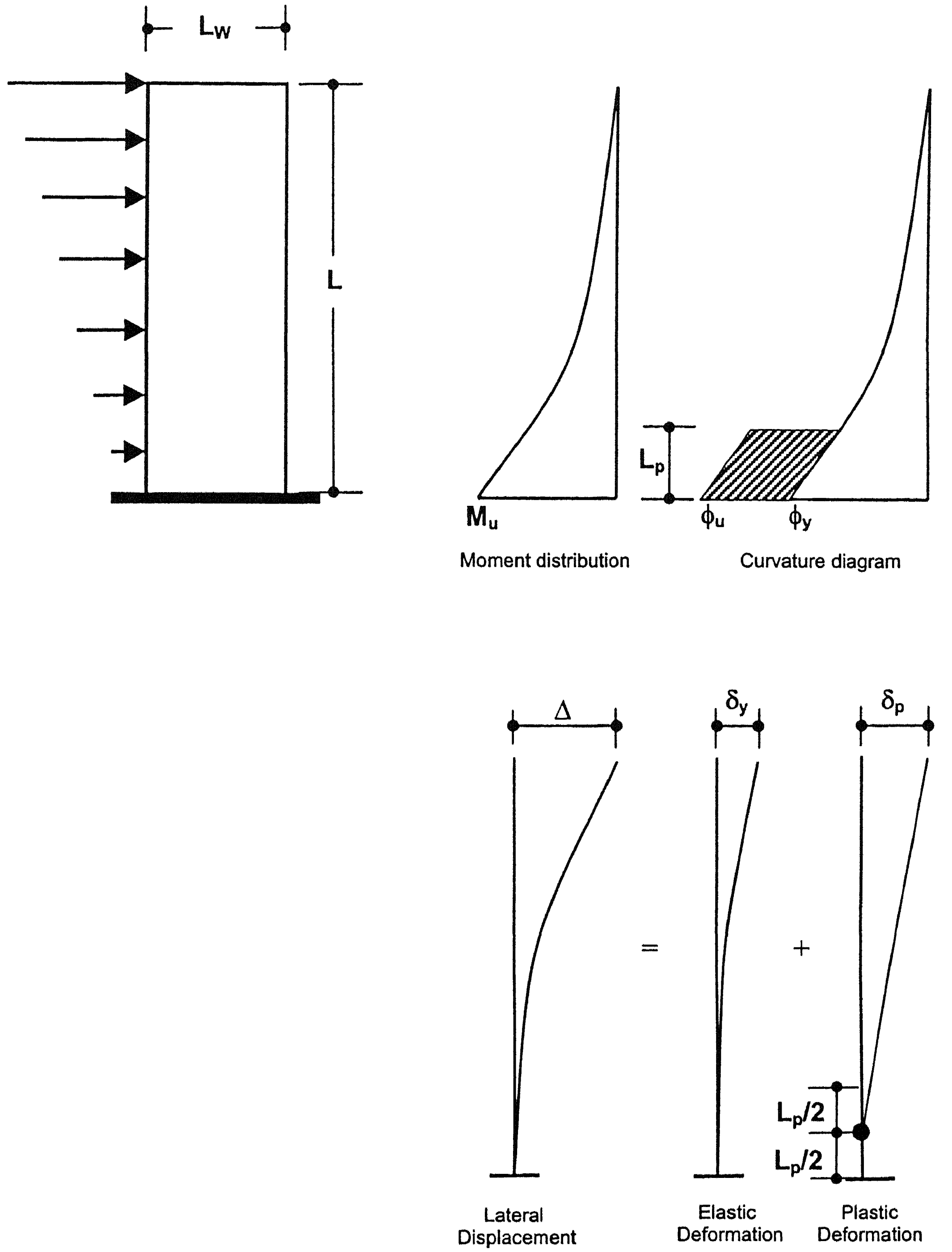



***Figure 3.2: Relationship between global and local deformations (from Wallace et al., 1993).***

# Chapter 4

# Results and Discussions

## 4.0 Introduction

The analysis procedure illustrated in the previous chapter has been carried out to study the performance of the example building, whose lateral force resisting system is primarily structural walls. A preliminary study conducted on the 3-storey building showed that the minimum wall dimensions and minimum reinforcement requirements govern the wall design for the considered dimensions of the building. Hence, the 3-storey building was not considered. Similarly, minimum codal provisions govern the wall design for six- and nine-storey buildings located in seismic zones III, II and I. Hence, the present study has been centered over the six- and nine-storey buildings located in seismic zones IV and V. The results of an earlier study by Navin (1993) are used for comparing the behaviour of structural wall buildings with those having moment resisting frames.

The materials used are M20 grade concrete and Fe415 grade steel. It is assumed that the buildings studied are at least one year old. The strength of the concrete increases with age and the design codes allow a factor of 1.2 to 1.25 on concrete strength as age factor for more than one-year-old concrete. Therefore, the characteristic cube strength of M20 concrete is taken as 25 kN/mm$^2$ in the evaluation of moment-curvature relations and a factor of 0.8 is used for converting the cube strength to cylinder strength.

In the calculation of tip deflection of wall, the shear deformations are neglected. Shear deformations have negligible influence on the ultimate deformation of walls with higher aspect ratio (height-to-width ratio). The walls studied in the present work have

high aspect ratio and hence the assumption is justified. P-Δ effect is also not considered in the present study.

Tables 4.1 and 4.2 give some interesting parameters for the walls considered. Figures 4.1 and 4.2 show these results in a graphical form. It is seen that the yield curvature does not vary much with seismic zone, number of storeys, or type of detailing. It may be recalled that all buildings had about the same wall length. Priestley and Kowalsky (1998) have observed that the yield curvature of walls strongly depends on the length of the wall and this is in line with the observation here. All the walls fail in compression mode implying that these were not *under-reinforced sections.* Despite failure in the compression zone, the walls exhibit considerable amount of ductility: the curvature ductility ranges from 3.6 to 12.2 and displacement ductility from 1.8 to 5.2. The overstrength factor ranges from 2.1 to 3.3, and the response reduction factor is in the range from 4.8 to 11.2. In case, the walls are designed to remain in the tension failure zone of the axial load-moment capacity curve, the ductility and response reduction factor will be higher.

## 4.1 Effect of Confinement

Different parameters for walls with boundary elements detailed as per IS: 13920-1993 divided by that for walls as per IS: 456-1978 are given in Table 4.3. The yield curvature, initial stiffness, yield displacement, and overstrength are not significantly affected by the type of detailing of the boundary elements. Detailing as per IS: 13920-1993 increases the ultimate curvature and the curvature ductility by about 30 to100%. Displacement ductility and drift capacity improve by about 20 to 70% if the boundary elements are detailed for ductility. Detailing as per IS:13920 leads to an increase in response reduction factor by about 25 to 70%.

## 4.2 Effect of Variation of Seismic Zone

Table 4.4 gives the ratio of different parameters for the building in seismic zone V to that for the building located in seismic zone IV. It is seen that ultimate curvature, curvature ductility, displacement ductility and drift capacity are higher in zone V as compared to that in zone IV. The ratios are higher for the six-storey building as compared to those for the nine-storey building. However, the overstrength is lower (15 to 23%) in zone V as compared to that in zone IV. This is in line with what has been observed earlier for frame buildings by Jain and Navin (1995). For a given building configuration, the relative significance of gravity loads as compared to seismic loads reduces as one goes to higher seismic zone. Hence, in high seismic zones the overstrength contributed by the load factors applied on gravity loads is lower as compared to that in low seismic zones. In zone V (as compared to zone IV), the ductility is higher and overstrength is lower; this means that no definite trend on respond reduction factor can be expected. Response reduction factor is higher in six-storey buildings and lower in nine-storey buildings as one goes from zone IV to zone V.

## 4.3 Effect of Number of Storeys

Ratio of different parameters for six-storey building versus nine-storey building are given in Table 4.5. In general, no definite trend is observed in ultimate curvature, ductility and drift. However, the overstrength for six-storey building is lower than that for the nine-storey building by about 15 to 25%. This trend is opposite of that observed for the frame buildings by Jain and Navin (1995). Here, as the number of storeys increases, the wall height increases and the seismic moment in the wall increases faster than the increase in gravity force on the wall. Hence, with increase in number of storeys, the relative significance of gravity loads in design reduces. Therefore, overstrength

contributed by the load factors applied on gravity loads is lower in buildings with a larger number of storeys. No definite trend is observed on the response reduction factors with respect to number of storeys.

## 4.4 Behaviour of Buildings with Walls *Versus* Frame Buildings

Figures 4.3 and 4.4 give the force-displacement relationship of six- and nine-storey buildings with walls and with frame systems. In case of walls, the curve corresponding to detailing as per IS:13920 is used. The curves for frames are taken from Jain and Navin (1995). It is seen that for six-storey building, the wall systems show more lateral stiffness as compared to the frame building. However, for the nine-storey building the lateral stiffness is quite comparable for the two types of systems. Overstrength of wall system is lower than that of frame systems in case of six-storey buildings while it is comparable for the nine-storey building. The drift level at which yielding takes place is lower for the six-storey wall buildings and comparable for the nine-storey buildings. The ultimate displacement and the drift capacity of the wall system is clearly lower than that of the frame building. Here it may be noted that the curves for walls are plotted till the ultimate point, while the frames were loaded only upto 2.5% drift and would take more lateral displacement than what is implied by figures 4.3 and 4.4. However, let us assume that 2.5% drift is the limit beyond which damage to the building will be too excessive and the ductility of frame building is assessed on the basis of 2.5% drift capacity. Even with this comparison, ductility of the wall building is lower than that of the frame building except for the case of six-storey building in seismic zone V.

## 4.5 Buildings with Dual System

The buildings may often have both walls and frames to resist lateral forces. The

codes describe such a building as a *dual system* provided the frames are designed to take at least 25% of the overall seismic design force. In such buildings, often the load distribution as per rigid floor diaphragm action leads to walls taking more than 75% of the lateral loads, and are designed for that load, while the design force for the frames is upgraded to 25% level. For instance, if the load distribution shows that the walls share 90% load and only 10% goes to the frames, the walls are designed for the 90% load while the frames are designed for 25% load.

Consider the example building but with a different arrangement of lateral load resisting system in the longitudinal direction. Let us say that the building has two walls and four frames to resist lateral loads, and such a building is to be designed as a dual system. Say the seismic coefficient for the building is 8%g, and the lateral load analysis shows that the walls share most of the lateral load. In that case, the two walls will receive about 8% weight of the building as the lateral load with each wall being designed for 4% of the building weight. The four frames put together will be designed for 2% of the building weight, with each frame receiving 0.5% of the building weight. Now consider the fact that the basic horizontal seismic coefficient for zone II (= 0.02) is one fourth of that for zone V (= 0.08). Similarly, the coefficient for zone I (= 0.01) is one fifth of that for zone IV (= 0.05). Hence, if the dual system building is designed in zone V, it will consist of zone V walls of this study and zone II frames of Jain and Navin (1995). Similarly, dual system building in zone IV will consist of zone IV walls and frames which are quite similar to zone I frames of Jain and Navin (1995). Figures 4.5 and 4.6 show the force-displacement relationships for the individual wall and individual frame of such a dual system.

The wall stiffness in each case is higher than that of the frames. For six-storey building, the wall-to-frame stiffness ratio is about 4.4 and 5.6, while that for the nine-

storey building is 2.7 and 3.2. For six-storey building, the walls yield while the frames are well within their elastic limit; at the time walls yield the frames carry lateral force which is only about 46% of their yield capacity. In case of nine-storey buildings, both walls and frames yield at around the same lateral displacement. When the walls yield, in case of six-storey building the four frames together carry about 20% of the total seismic force. This value is about 28% in case of nine-storey building.

• • •

*Table 4.1 Calculated properties of the structural walls designed for six-storey building in zone IV and zone V at sectional level and global level.*

| Parameter | Zone V | | Zone IV | |
| --- | --- | --- | --- | --- |
| | Confinement reinforcement details in boundary element | | | |
| | According to IS:13920-1993 | According to IS:456-1978 | According to IS:13920-1993 | According to IS:456-1978 |
| Initial Stiffness (kN/m) | 30,750 | 28,600 | 20,500 | 20,250 |
| Yield Curvature, $\phi_y$ ($mm^{-1}$) | $6.44\times10^{-7}$ | $6.60\times10^{-7}$ | $7.01\times10^{-7}$ | $6.89\times10^{-7}$ |
| Ultimate Curvature, $\phi_u$ ($mm^{-1}$) | $7.90\times10^{-6}$ | $4.42\times10^{-6}$ | $3.33\times10^{-6}$ | $2.50\times10^{-6}$ |
| Curvature Ductility, $\mu_\phi$ | 12.2 | 6.7 | 4.7 | 3.62 |
| Yield Displacement, $\Delta_y$ (mm) | 73.6 | 75.5 | 80.2 | 78.9 |
| Plastic Displacement, $\Delta_p$ (mm) | 310.0 | 159.5 | 110.5 | 76.7 |
| Total Displacement, $\Delta_{tot}$ (mm) | 384.1 | 235.1 | 190.7 | 155.6 |
| Displacement Ductility, $\mu_\Delta$ | 5.2 | 3.11 | 2.4 | 1.97 |
| Drift Capacity, in % of H | 1.9 | 1.2 | 0.9 | 0.7 |
| Maximum Moment, $M_{max}$ (kN-m) | 35,700 | 34,500 | 26,300 | 25,500 |
| Failure Mode | Comp.* | Comp.* | Comp.* | Comp.* |
| Overstrength, $\Omega$ | 2.15 | 2.10 | 2.54 | 2.46 |
| $R=\mu_\Delta\Omega$ | 11.2 | 6.5 | 6.1 | 4.8 |

* Compression failure: Concrete in top boundary element reaches the ultimate strain.

*Table 4.2 Calculated properties of the structural walls designed for nine-storey building in zone IV and zone V at sectional level and global level.*

| Parameter | Zone V | | Zone IV | |
|---|---|---|---|---|
| | Confinement reinforcement details in boundary element | | | |
| | According to IS:13920-1993 | According to IS:456-1978 | According to IS:13920-1993 | According to IS:456-1978 |
| Initial Stiffness, (kN/m) | 18,300 | 18,400 | 14,400 | 14,550 |
| Yield Curvature, $\phi_y$ ($mm^{-1}$) | $6.67\times10^{-7}$ | $6.45\times10^{-7}$ | $6.93\times10^{-7}$ | $6.62\times10^{-7}$ |
| Ultimate Curvature, $\phi_u$ ($mm^{-1}$) | $5.44\times10^{-6}$ | $3.15\times10^{-6}$ | $5.01\times10^{-6}$ | $2.46\times10^{-6}$ |
| Curvature Ductility, $\mu_\phi$ | 8.1 | 4.8 | 7.4 | 3.7 |
| Yield Displacement, $\Delta_y$ (mm) | 165.1 | 159.6 | 170.7 | 164.0 |
| Plastic Displacement, $\Delta_p$ (mm) | 364.1 | 190.8 | 335.9 | 137.0 |
| Total Displacement, $\Delta_{tot}$ (mm) | 529.2 | 350.5 | 506.9 | 301.0 |
| Displacement Ductility, $\mu_\Delta$ | 3.2 | 2.2 | 2.9 | 1.8 |
| Drift Capacity, in % of H | 1.76 | 1.17 | 1.68 | 1.00 |
| Maximum Moment, $M_{max}$ (kN-m) | 69,600 | 67,800 | 56,500 | 55,000 |
| Failure Mode | Comp.* | Comp.* | Comp.* | Comp.* |
| Overstrength, $\Omega$ | 2.56 | 2.48 | 3.33 | 3.23 |
| $R = \mu_\Delta \Omega$ | 8.2 | 5.4 | 9.6 | 5.8 |

* Compression failure: Concrete in top boundary element reaches the ultimate strain.

***Table 4.3 Ratio of different parameters of structural walls with confinement in boundary element as per IS: 13920-1993 to the IS: 456-1978***

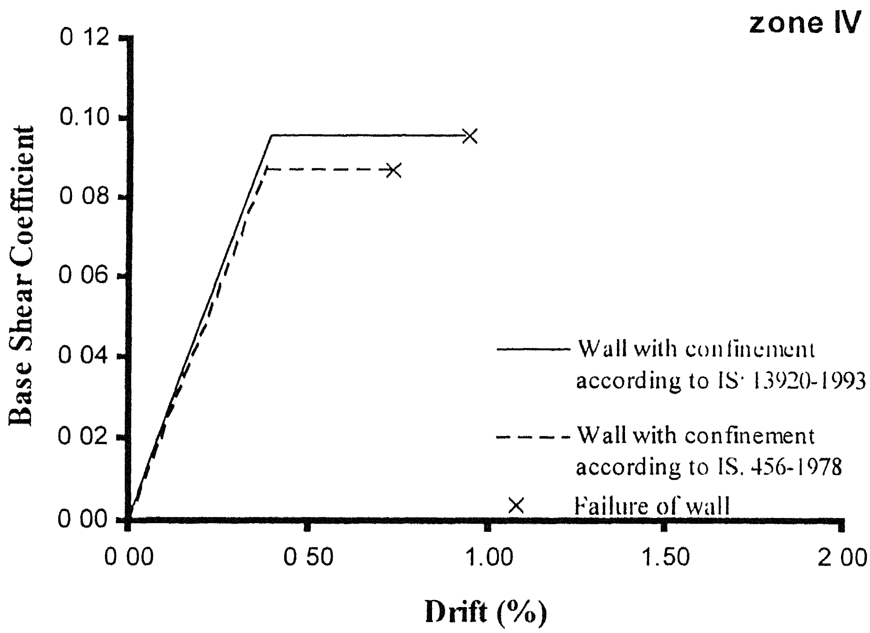
| Parameter | Six-storey | | Nine-storey | |
|---|---|---|---|---|
| | Zone V | Zone IV | Zone V | Zone IV |
| Initial Stiffness | 1.07 | 1.01 | 0.99 | 0.99 |
| Yield Curvature, $\phi_y$ | 0.98 | 1.02 | 1.03 | 1.05 |
| Ultimate Curvature, $\phi_u$ | 1.79 | 1.33 | 1.73 | 2.04 |
| Curvature Ductility, $\mu_\phi$ | 1.82 | 1.31 | 1.69 | 1.98 |
| Displacement Ductility, $\mu_\Delta$ | 1.67 | 1.22 | 1.45 | 1.58 |
| Drift Capacity, in % of H | 1.63 | 1.29 | 1.50 | 1.68 |
| Overstrength, $\Omega$ | 1.02 | 1.03 | 1.03 | 1.03 |
| $R=\mu_\Delta\Omega$ | 1.70 | 1.25 | 1.52 | 1.63 |

***Table 4.4 Ratio of different parameters of structural walls in zone V to zone IV***

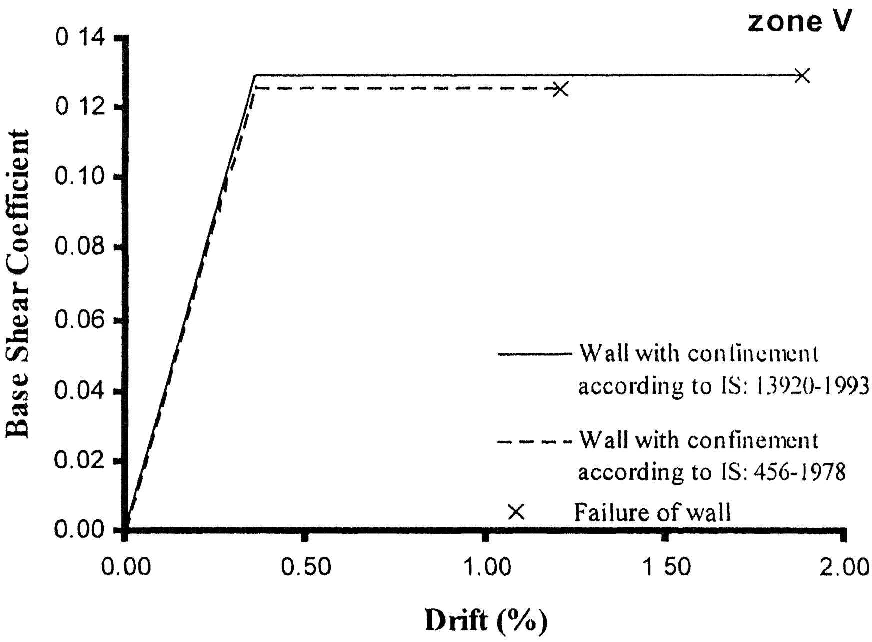
| Parameter | IS: 13920-1993 | | IS: 456-1978 | |
|---|---|---|---|---|
| | Six-storey | Nine-storey | Six-storey | Nine-storey |
| Ultimate Curvature, $\phi_u$ | 2.37 | 1.09 | 1.77 | 1.28 |
| Curvature Ductility, $\mu_\phi$ | 2.57 | 1.10 | 1.85 | 1.29 |
| Displacement Ductility, $\mu_\Delta$ | 2.17 | 1.10 | 1.58 | 1.20 |
| Drift Capacity, in % of H | 2.00 | 1.05 | 1.58 | 1.17 |
| Overstrength, $\Omega$ | 0.85 | 0.77 | 0.85 | 0.77 |
| $R=\mu_\Delta\Omega$ | 1.85 | 0.85 | 1.35 | 0.92 |

***Table 4.5 Ratio of different parameters of structural walls of six-storey to nine-storey***

| Parameter | IS: 13920-1993 | | IS: 456-1978 | |
|---|---|---|---|---|
| | Zone V | Zone IV | Zone V | Zone IV |
| Ultimate Curvature, $\phi_u$ | 1.45 | 0.66 | 1.40 | 1.02 |
| Curvature Ductility, $\mu_\phi$ | 1.51 | 0.65 | 1.40 | 0.98 |
| Displacement Ductility, $\mu_\Delta$ | 1.63 | 0.83 | 1.41 | 1.08 |
| Drift Capacity, in % of H | 1.07 | 0.56 | 0.98 | 0.73 |
| Overstrength, $\Omega$ | 0.84 | 0.76 | 0.85 | 0.76 |
| $R=\mu_\Delta\Omega$ | 1.35 | 0.63 | 1.20 | 0.81 |

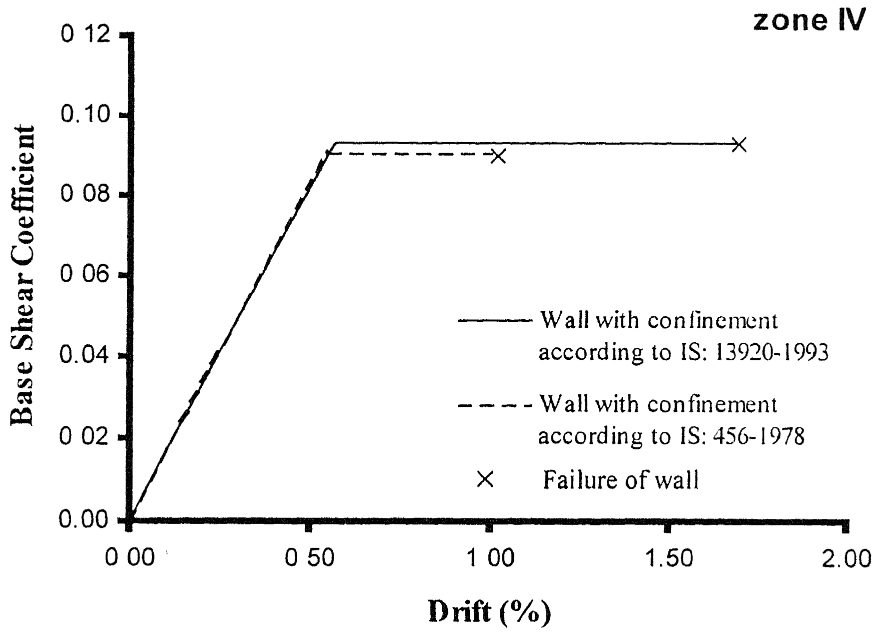



**(a)**

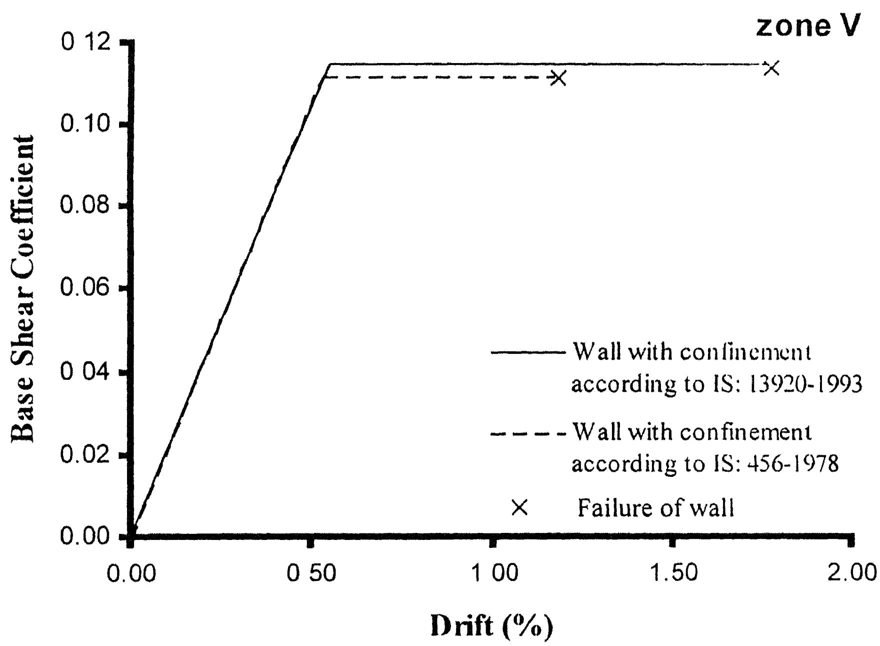



**(b)**

***Figure 4.1 Comparison of force displacement relationship of six-storey building with structural walls as lateral forces resisting system (with boundary elements, confined according to IS: 13920-1993 and IS: 456-1978) (a) in seismic zone IV (b) in seismic zone V***

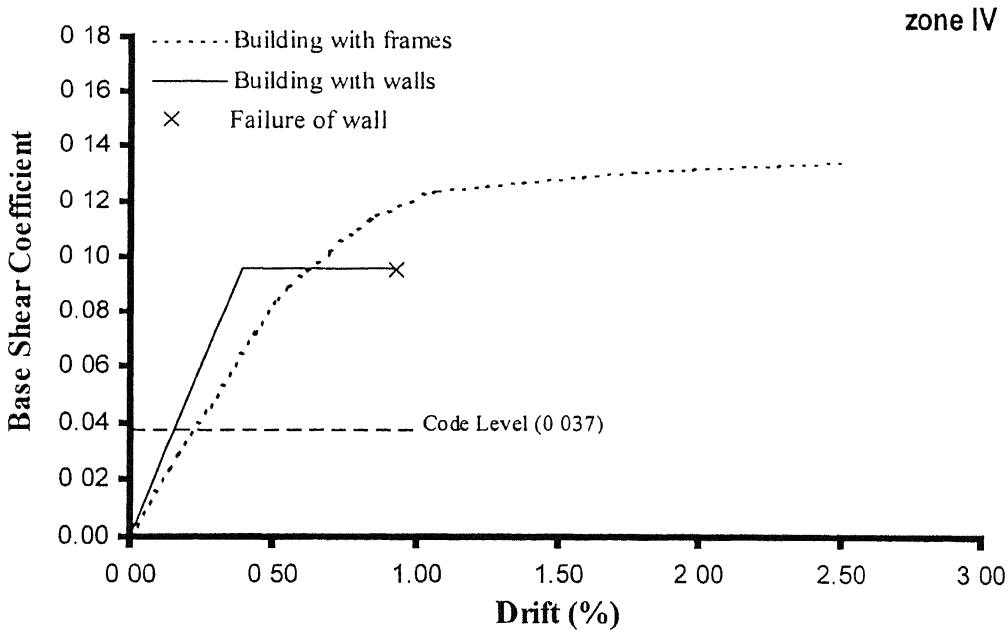



(a)

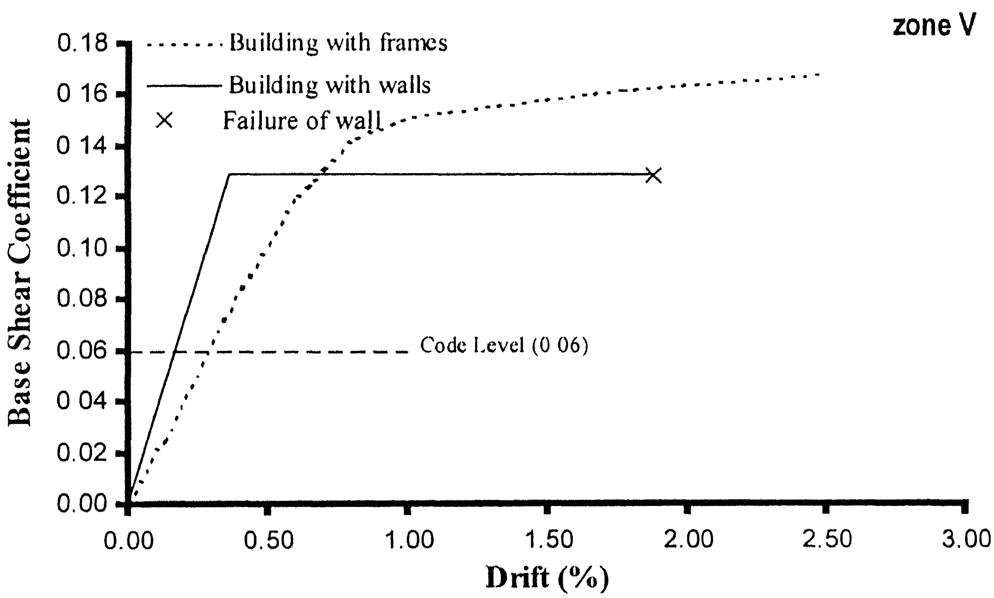



(b)

***Figure 4.2** Comparison of force displacement relationship of nine-storey building with structural walls as lateral forces resisting system (with boundary elements, confined according to IS: 13920-1993 and IS: 456-1978) (a) in seismic zone IV (b) in seismic zone V*

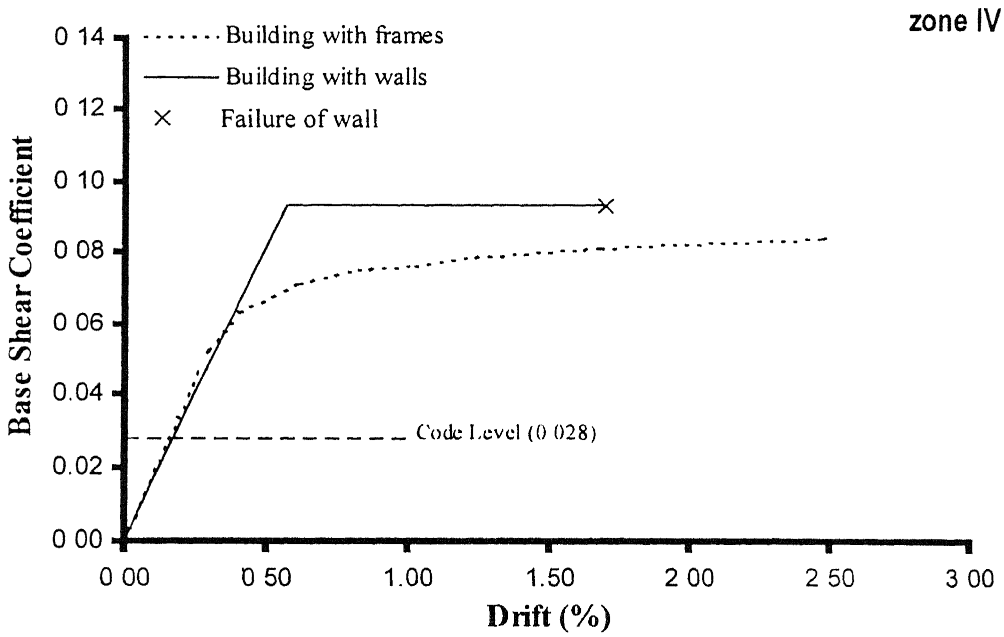



(a)

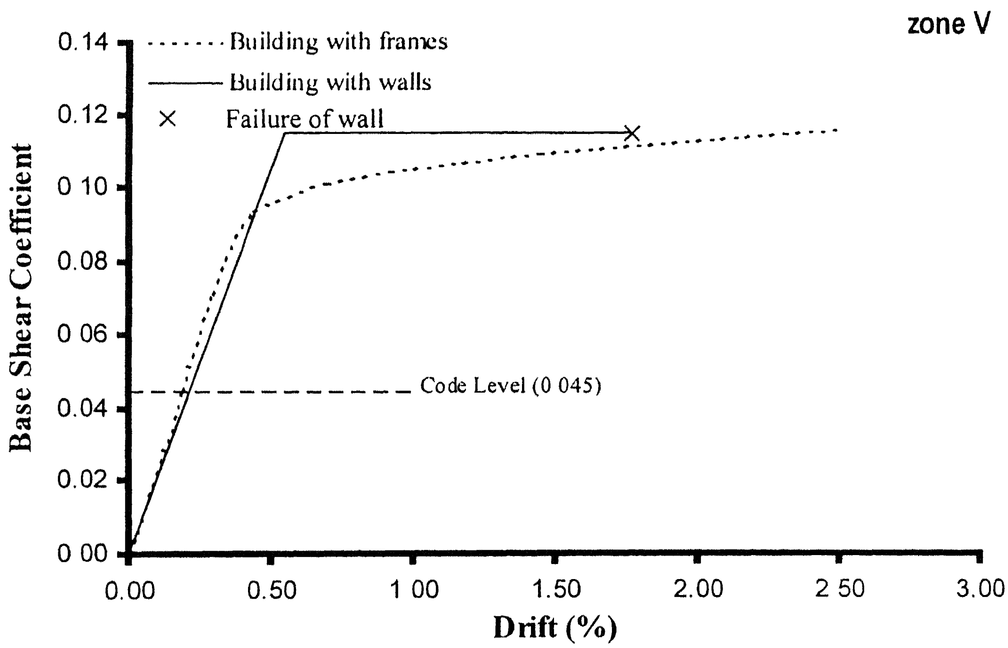



(b)

***Figure 4.3** Comparison of force displacement relationship of six-storey building with structural walls and moment resisting frames as lateral force resisting system (a) in seismic zone IV (b) in seismic zone V*

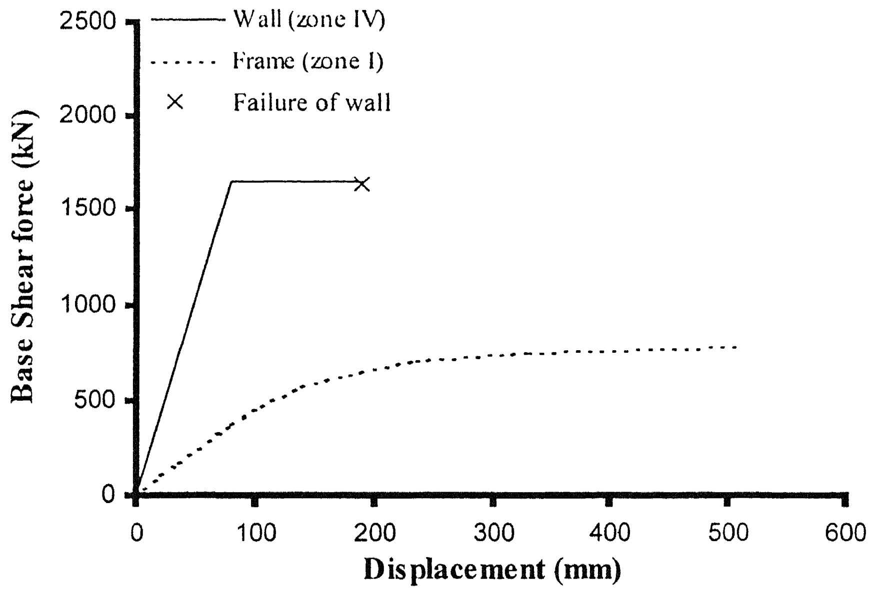



(a)

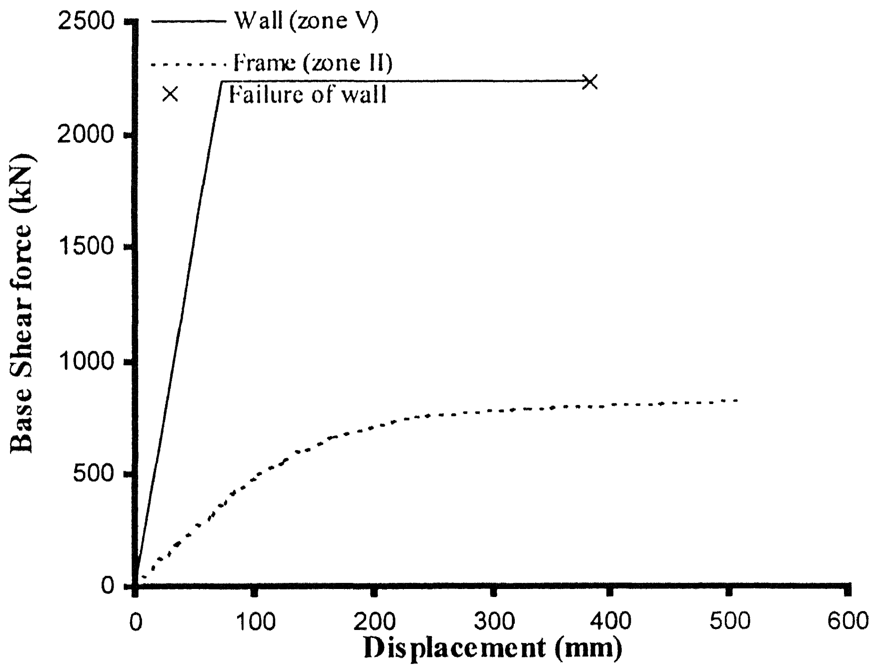



(b)

***Figure 4.4 Comparison of force displacement relationship for nine-storey building with structural walls and moment resisting frames as lateral force resisting system (a) in seismic zone IV (b) in seismic zone V***

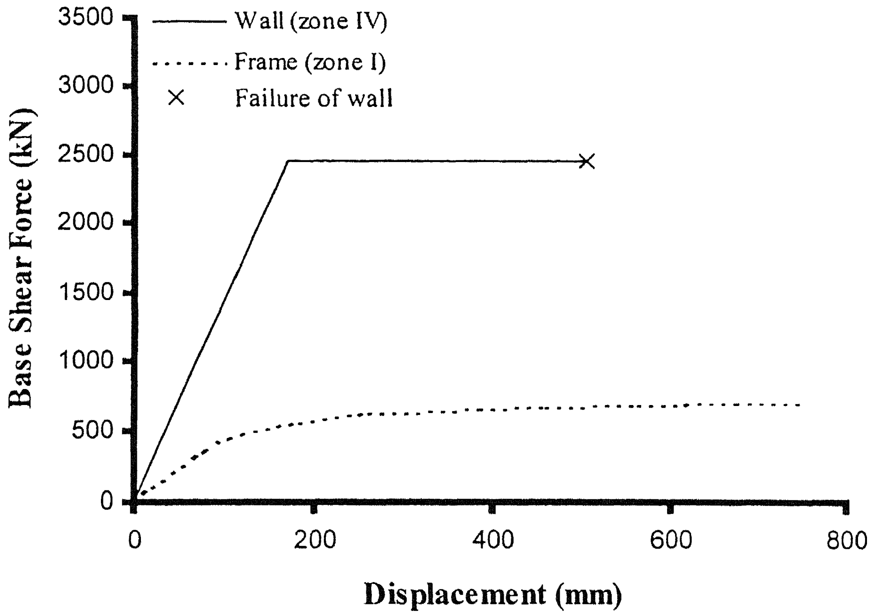



(a)

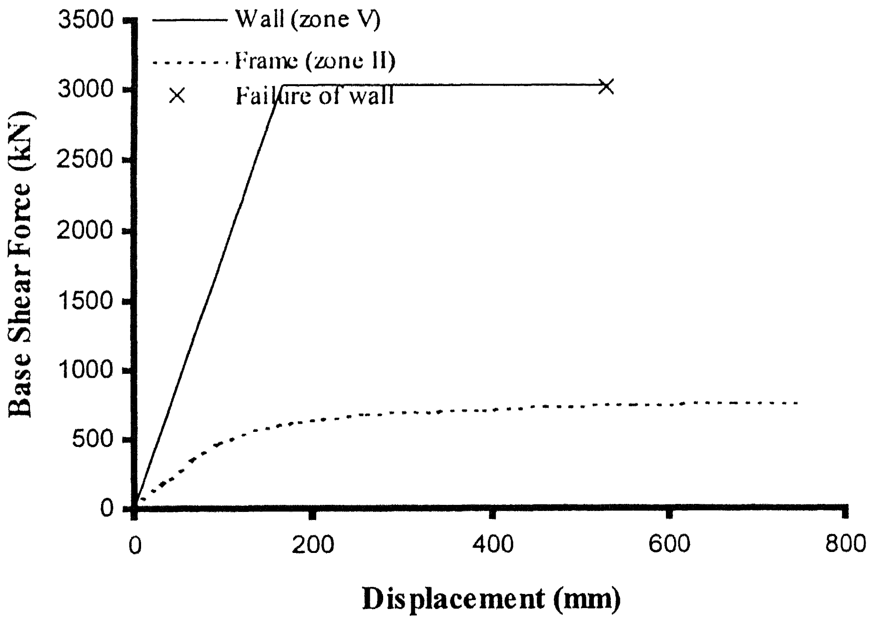



(b)

***Figure 4.5** Comparison of force displacement relationship of (a) six-storey building zone IV wall with six-storey building zone I frame (b) six-storey building zone V wall with six-storey building zone II frame*




**(a)**




**(b)**

***Figure 4.6** Comparison of force displacement relationship of (a) nine-storey building zone IV wall with nine-storey building zone I frame (b) nine-storey building zone V wall with nine-storey building zone II frame*

# Chapter 5

# Summary and Conclusions

A computer program has been developed to determine the moment curvature relationship of reinforced concrete structural walls with confined boundary elements. The program takes into account confinement of concrete in the boundary elements and adopts different concrete model for the unconfined part of wall (web) and the confined part of wall (boundary elements). The program gives the sectional properties like the yield curvature and ultimate curvature of the section, by which the curvature ductility of the section can be calculated. The local deformations of the wall are used to evaluate the global deformations of the wall.

An example building is designed as per the Indian codes (IS: 1893-1984, IS: 13920-1993 and IS: 456-1978) with lateral force resisting system consisting of structural walls. The building plan is similar to that of the building studied by Jain and Navin (1995). The building is designed for seismic zones IV and V. Number of stories considered are six and nine. The boundary elements are detailed in two ways: as per IS: 13920-1993 with closely-spaced shear stirrups, and as per IS: 456-1978 with only nominal shear stirrups. Using the computer program developed here the sectional properties of the walls are evaluated and these are used to study the global behaviour of the building in terms of force displacement relationship. The effect of the confinement, seismic zone, and number of storeys on the ductility capacity, maximum drift-limit, overstrength, and response reduction factor has been studied.

It is seen that the initial stiffness, yield displacement and overstrength are insensitive to the concrete confinement in the boundary elements. The strong dependency

of yield curvature on the length of the wall, irrespective of the percentage of the reinforcement in the web and the confinement in boundary elements, is also seen. It was also observed that the ultimate curvature of the walls, and hence the curvature ductility, depends on the confinement reinforcement in the boundary elements. Curvature ductility increases by 30% to 100% when the walls are detailed as per IS: 13920-1993 as against IS:456-1978. Displacement ductility and drift capacity improve by about 20% to 70% and the response reduction factor by about 25% to 70%.

Overstrength of the walls in seismic zone IV is found to be higher (15% to 23%) than that in seismic zone V. No definite trend was observed in the variation of response reduction factor with respect to seismic zone.

The overstrength of the walls of nine-storey building is higher than that of six-storey building by 15% to 25%. This is in contradiction with the behaviour of the frames, where the overstrength associated with low-rise frames is higher than that of the high-rise frames. There is no particular trend in the variation of response reduction factor with respect to number of stories.

Lateral response of the building with walls was compared with that of the similar building having moment resisting frames. It is seen that in six-storey building the wall buildings are stiffer than the frame buildings and in nine-storey building the lateral stiffness of both the systems is about the same. It is also observed that the overstrength of wall systems is lower in case of six-storey buildings while it is comparable for nine-storey buildings. The drift at which yielding takes place is lower for the six-storey wall buildings and it is comparable for the nine-storey buildings. The ultimate displacement and drift capacity of the wall systems is found to be less than those of the frame systems.

A study has been done on the example building having dual system in which the walls are designed for the full lateral forces and the frames are designed for 25% of the

total lateral loads. It is seen that the wall stiffness is 4.4 and 5.6 times higher than that of the frames in six-storey building, and it is 2.7 and 3.2 times in case of nine-storey building. In six-storey buildings, when walls reach yield point the frames are still in elastic range (46% of the yield capacity). Both walls and frames yield around the same displacement in the nine-storey buildings. When the walls yield, all four frames together carried about 20% of the total seismic force in case of six-story building and about 28% in case of nine-storey building.

This study clearly underlines the importance of providing closely-spaced stirrups in the boundary elements of shear walls. This results in only very nominal increase in the reinforcement consumption while there is substantial increase in the response reduction factor. This implies better performance of the building in case of strong earthquake shaking. Alternatively, it implies lower seismic design force leading to economy in construction. The results indicate that it will be reasonable to reduce the design seismic force by about 35% whenever the boundary elements have special confinement.

In the current study the walls are assumed to be rigidly held at the base, however walls are subjected to large bending moment at its base as a result of which the footing supporting the wall tend to rotate depending on the soil foundation system. This will have significant effect in the roof displacement of the wall and must be accounted for in analysis. In addition, it is desirable to also consider the shear deformations in the wall.

It is well recognised that the shear walls provide a very reliable structural system as a protection against collapse caused by strong earthquake shaking. However, there have been concerns about the ductility and other characteristics of shear walls. This study is a small step in this direction and more thorough studies are needed in order to fully exploit the advantages of shear wall systems in our design process.

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Appendix A

# Stress-Strain Equations for Confined and Unconfined Concrete

## A.1 Confined Concrete

The stress-strain relations for unconfined concrete proposed by Saatcioglu and Razvi (1992) can be divided into three regions (Figure 2.1): Parabolic ascending part and a linear portion of the descending branch, followed by a constant residual strength at 20% strength.

(a) Parabolic ascending branch is characterised by equation A-1, which becomes identical to the one proposed by Hognestad et al. (1951) for unconfined concrete.

$$f_c = f'_{cc}\left[\frac{2\varepsilon_c}{\varepsilon_1} - \left(\frac{\varepsilon_c}{\varepsilon_1}\right)^2\right]^{-(1+2K)} \leq f'_{cc} \qquad for \quad 0 \leq \varepsilon_c \leq \varepsilon_1 \qquad \text{A-1}$$

(b) Second part is a linearly descending branch defined with a strain corresponding to 85% of peak stress, and is given by

$$f_c = f'_{cc} - \frac{\varepsilon_c - \varepsilon_1}{\varepsilon_{85} - \varepsilon_1} 0.15 f'_{cc} \leq 0.2 f'_{cc} \qquad for \quad \varepsilon_1 \leq \varepsilon_c \leq \varepsilon_{20} \qquad \text{A-2}$$

(c) A constant residual strength of 20% strength level is assumed beyond the linear descending part.

$$f_c = 0.20 f'_{cc} \qquad for \quad \varepsilon_c \geq \varepsilon_{20} \qquad \text{A-3}$$

where,

$f'_{cc} = f'_{co} + k_1 f_{le}$

$f'_{cc}$ and $f'_{co}$ = strength of confined and unconfined concrete,

$K = k_1 f_{le} / f'_{co}$

$k_l = 6.7(f_{le})^{-0.17}$

$f_{le}$ = equivalent uniform pressure (in MPa) given by; $f_{le} = k_2 f_l$

$$f_{le} = \frac{f_{lex}\, b_{cx} + f_{ley}\, b_{cy}}{b_{cx} + b_{cy}} \text{ (for rectangular section)}$$

$f_{lex}$, $f_{ley}$ = effective lateral pressures acting perpendicular to core dimensions $b_{cx}$ in the x-direction and $b_{cy}$ in the y-direction, respectively ,

$$k_2 = 0.26\sqrt{\left[\left(\frac{b_c}{s}\right)\left(\frac{b_c}{s_l}\right)\left(\frac{1}{f_l}\right)\right]} \leq 1.0$$

$b_c$ = core dimension of square section,

$s_l$ = spacing between laterally supported longitudinal reinforcement,

$s$ = center to center spacing of transverse reinforcement,

$f_l$ = uniform confining pressure (in MPa) given by; $f_l = \dfrac{\sum A_s\, f_{yh} \sin(\alpha)}{s\, b_c}$

$A_s$, $f_{yh}$ = area and yield strength of the transverse reinforcement, respectively,

$\alpha$ = angle between the transverse reinforcement and $b_C$, and is equal to $90^0$ if the reinforcement is perpendicular to $b_C$,

$\varepsilon_1 = \varepsilon_{01}(1 + 5K)$

$\varepsilon_{85} = 260\,\rho\,\varepsilon_1 + \varepsilon_{085}$

$\varepsilon_{01}$, $\varepsilon_{085}$ = strain corresponding to peak stress and 85% of peak stress for unconfined concrete. In the absence of test data $\varepsilon_{01}$ may be taken as 0.002 and $\varepsilon_{085}$ as 0.0038, for slow rate of loading,

$$\rho = \frac{\sum A_s}{s\left(b_{cx} + b_{cy}\right)}$$

$b_{cx}$, $b_{cy}$ = core dimensions of rectangular section,

$f_c$ = stress corresponding to a strain of $\varepsilon_c$ from the stress-strain relation,

$\varepsilon_1$, $\varepsilon_{85}$ = strain corresponding to peak stress and 85% of the peak stress, respectively, for confined concrete, and

$\varepsilon_{20}$ = strain at 20% of maximum stress on the descending branch of the stress-strain curve of confined concrete.

## A.2 Unconfined Concrete

IS:456 (1978) model concrete stress-strain curve relationship as a parabolic-rectangle. The parabolic part is represented by a second degree parabola with its apex at a strain of 0.002 followed by a straight line at the maximum stress level upto a strain of 0.0035 (Figure 2.2). The parabolic portion of the curve is defined by the equation proposed by Hognestad et al. (1955) and is given as

$$f_c = f_c''\left[\frac{2\varepsilon_c}{\varepsilon_0} - \left(\frac{\varepsilon_c}{\varepsilon_0}\right)^2\right] \qquad \text{A-4}$$

where,

$\varepsilon_c$ = stress at any point,

$\varepsilon_0$ = strain at which parabolic part showing elastic region ends and the straight line representing the plastic region starts. IS:456 (1978) assumes $\varepsilon_0$ as 0.002, and

$f_c''$ = maximum stress corresponding to $\varepsilon_0$.

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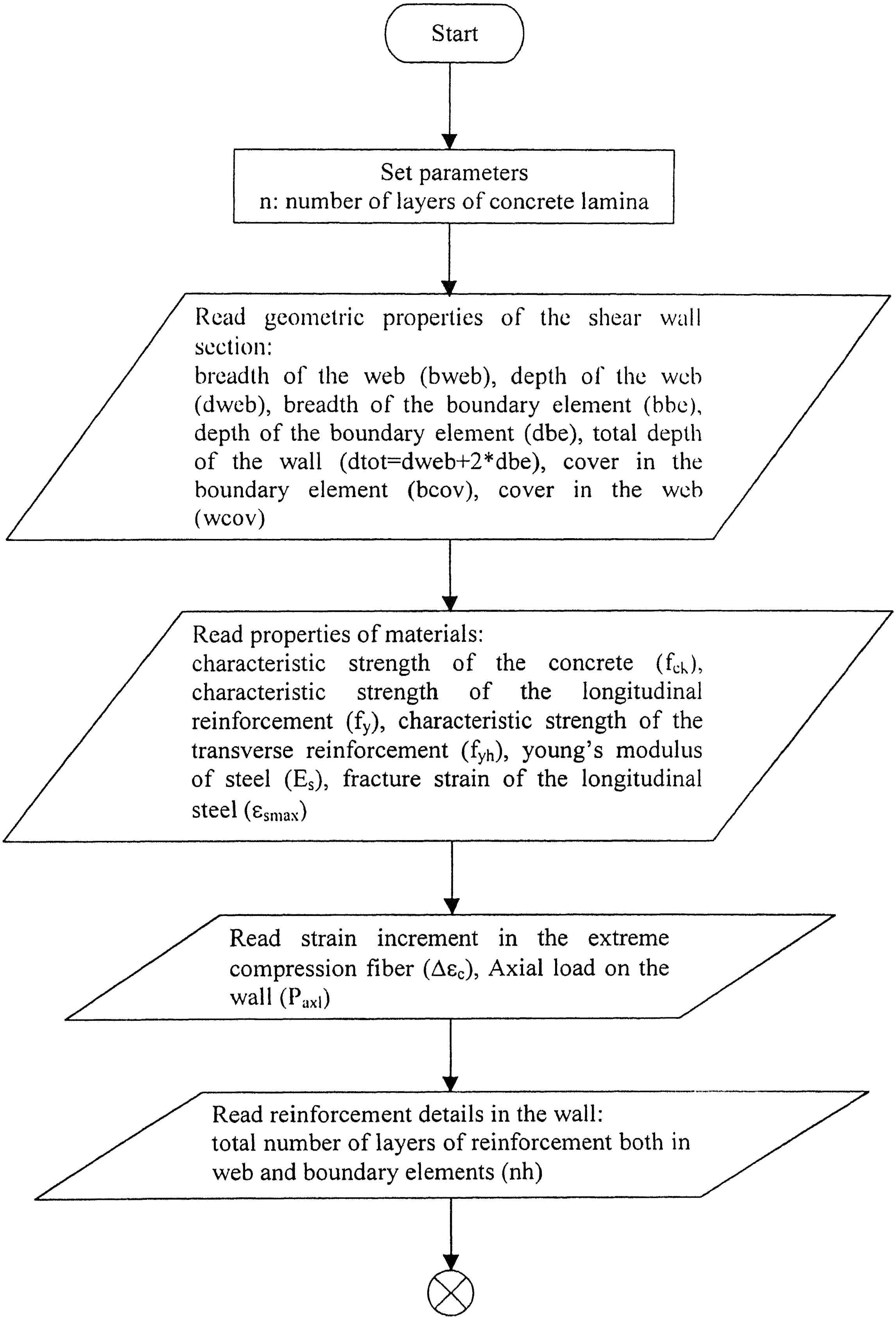
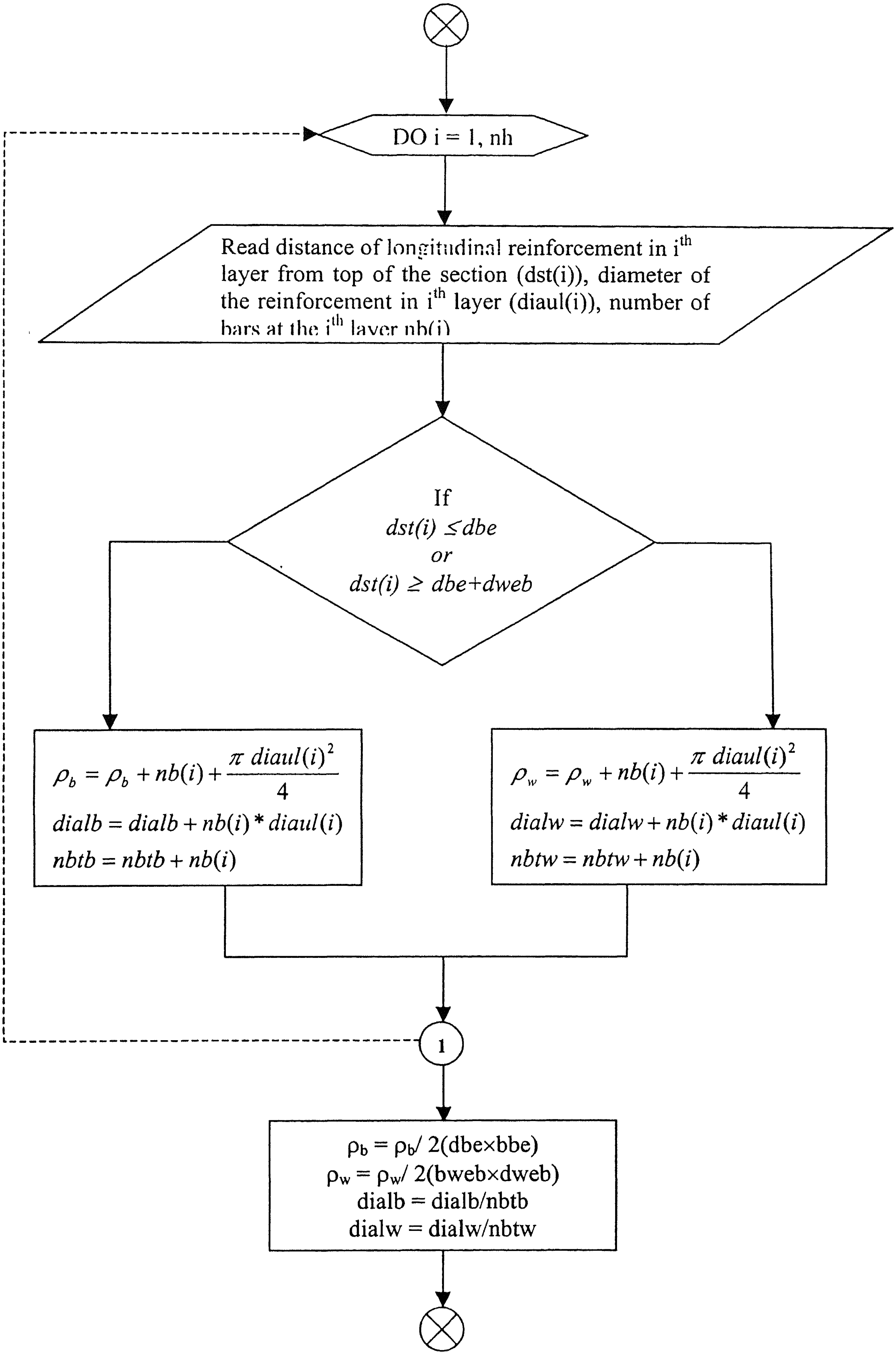
## Appendix B

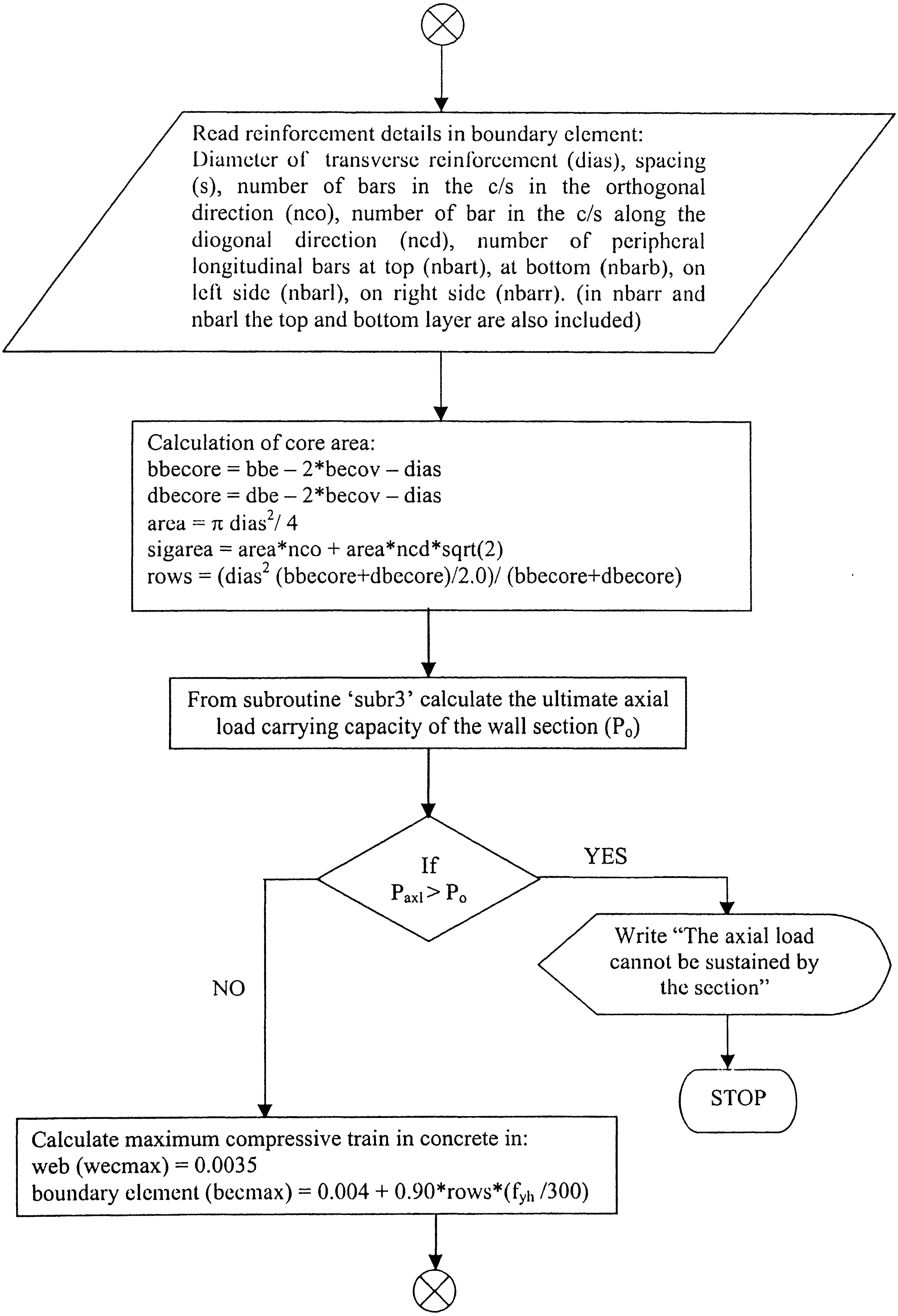
# Flow Chart and Program Listing

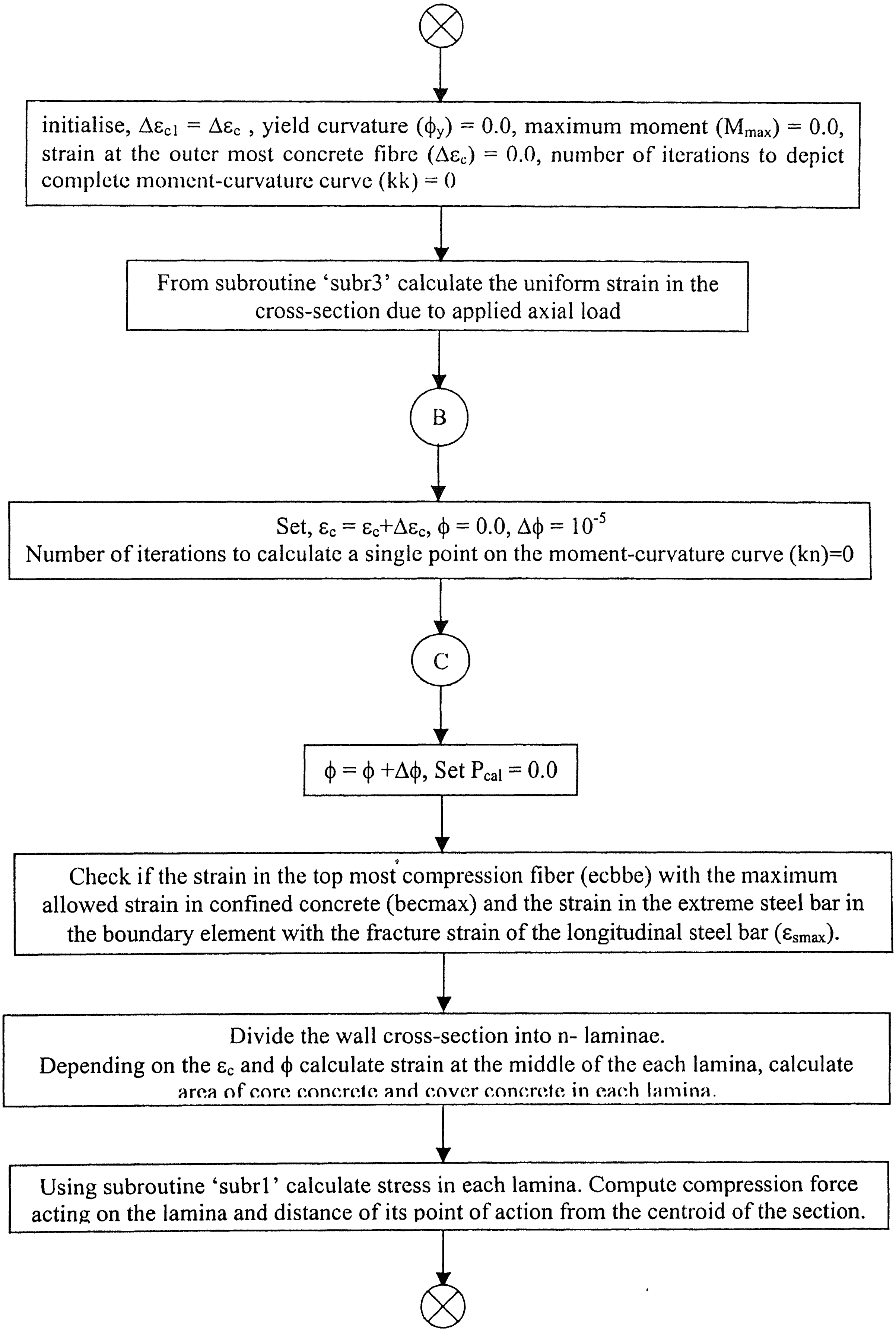
A computer program has been developed to determine the moment-curvature relationship of a reinforced concrete structural wall with confined boundary elements. From the moment-curvature curve yield curvature, ultimate curvature and curvature ductility of the section can be determined as explained in chapter 2. The stress-strain curve for unconfined and confined concrete has been taken as per IS: 456-1978 and Saatcioglu and Razvi (1992), respectively. The stress-strain curve for steel has been taken as per IS: 456-1978. However, any other stress-strain curve can be easily incorporated by changing the subroutine 'subr1' and 'subr2'.

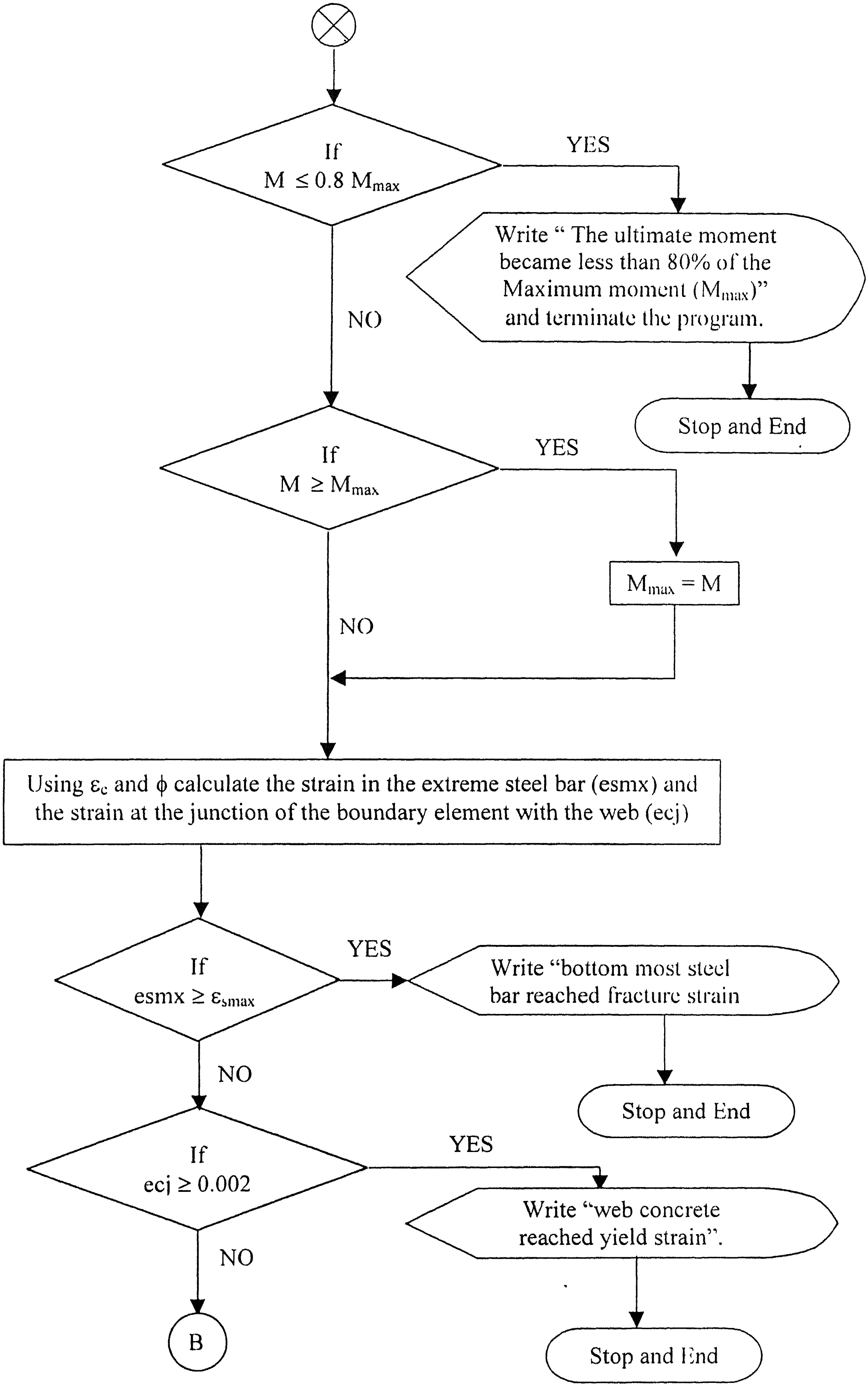
The detailed flow chart of the computer program is presented herein. The complete program listing is also provided.

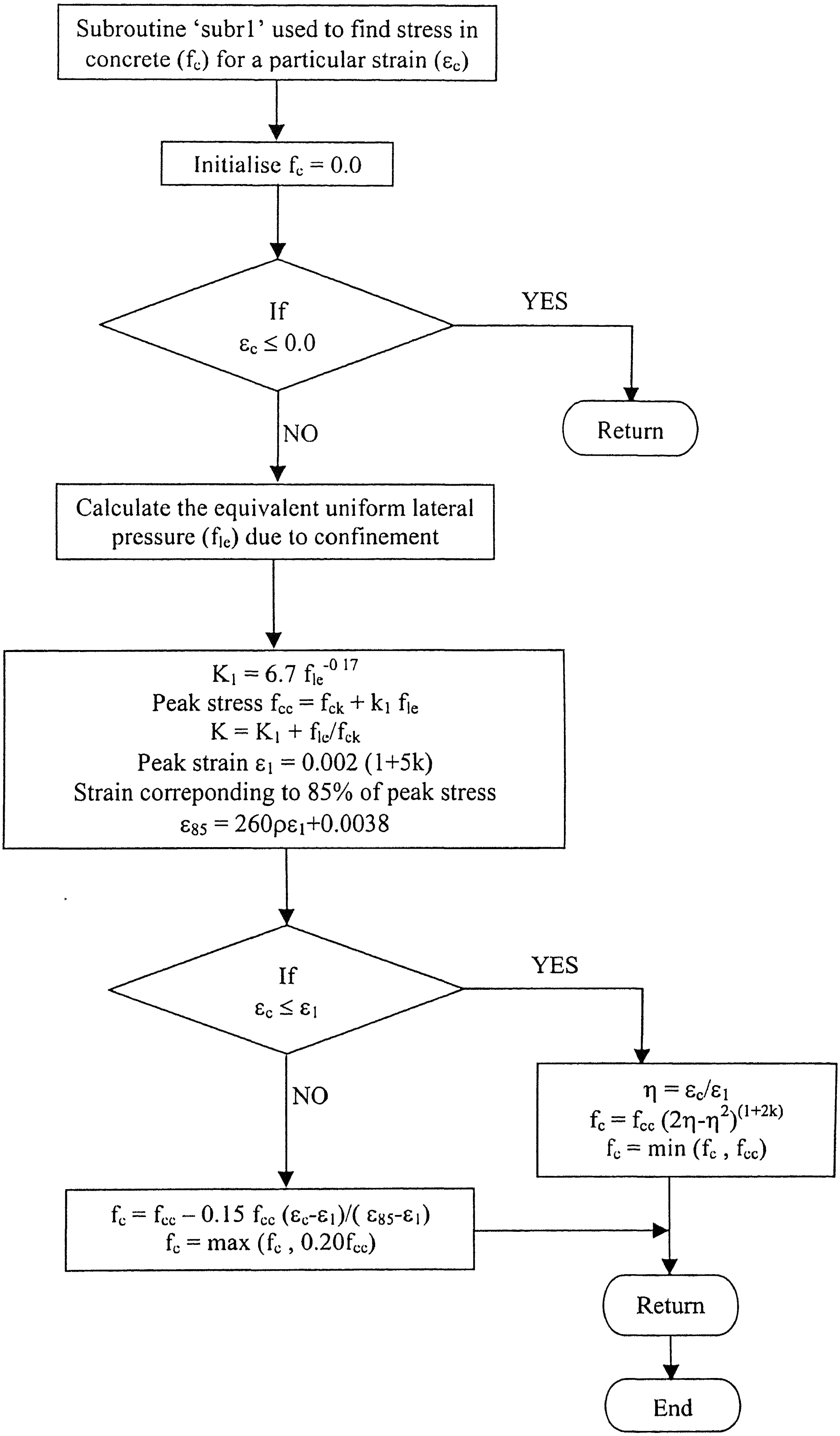

Start
Set parameters
n: number of layers of concrete lamina
Read geometric properties of the shear wall section:
breadth of the web (bweb), depth of the web (dweb), breadth of the boundary element (bbe), depth of the boundary element (dbe), total depth of the wall (dtot=dweb+2*dbe), cover in the boundary element (bcov), cover in the web (wcov)
Read properties of materials:
characteristic strength of the concrete ($f_{ck}$), characteristic strength of the longitudinal reinforcement ($f_y$), characteristic strength of the transverse reinforcement ($f_{yh}$), young's modulus of steel ($E_s$), fracture strain of the longitudinal steel ($\varepsilon_{smax}$)
Read strain increment in the extreme compression fiber ($\Delta\varepsilon_c$), Axial load on the wall ($P_{axl}$)
Read reinforcement details in the wall:
total number of layers of reinforcement both in web and boundary elements (nh)

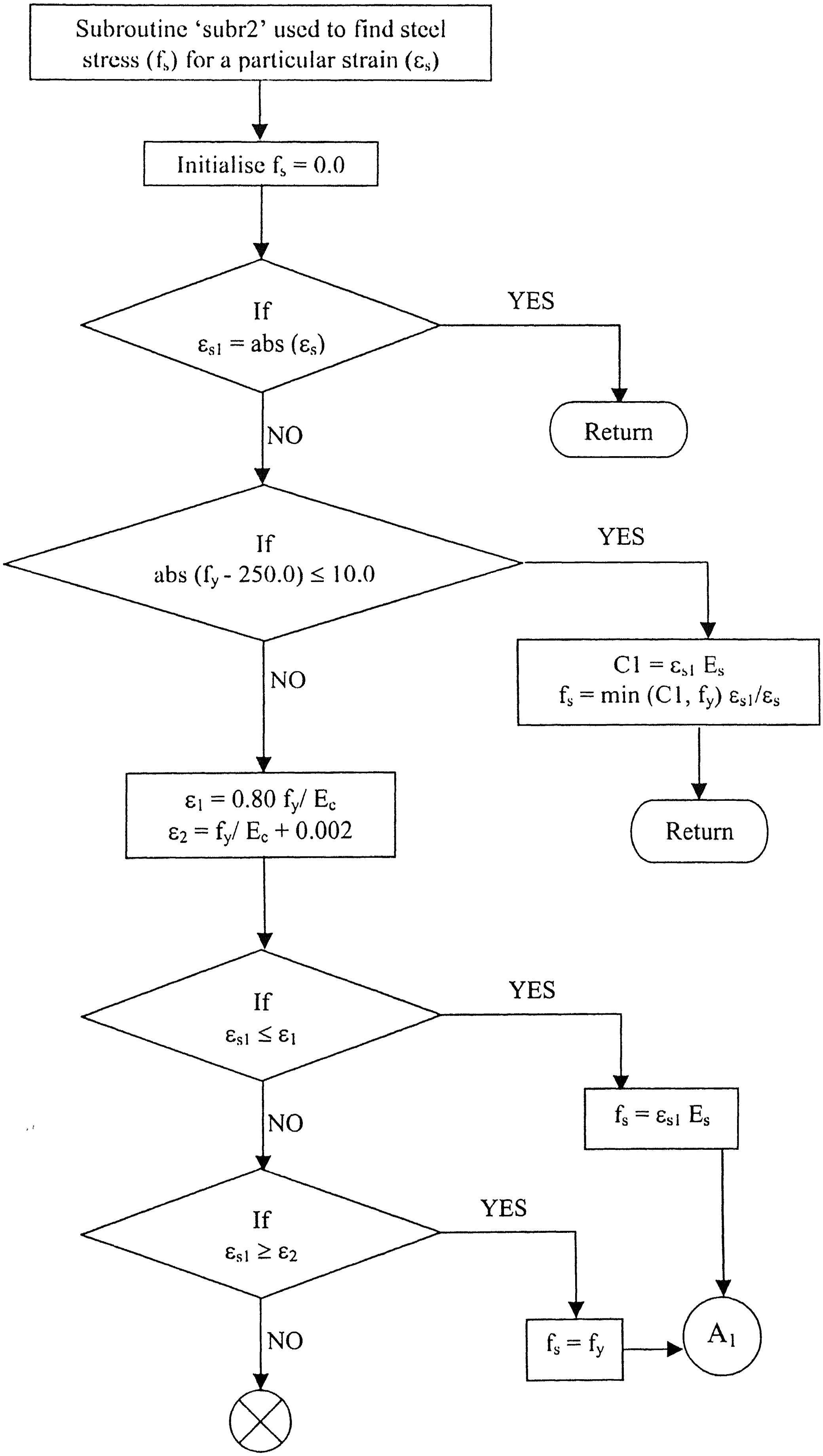

DO i = 1, nh
Read distance of longitudinal reinforcement in $i^{th}$ layer from top of the section (dst(i)), diameter of the reinforcement in $i^{th}$ layer (diaul(i)), number of bars at the $i^{th}$ layer nb(i)
If
$dst(i) \leq dbe$
or
$dst(i) \geq dbe+dweb$
$\rho_b = \rho_b + nb(i) + \frac{\pi\ diaul(i)^2}{4}$
$dialb = dialb + nb(i) * diaul(i)$
$nbtb = nbtb + nb(i)$
$\rho_w = \rho_w + nb(i) + \frac{\pi\ diaul(i)^2}{4}$
$dialw = dialw + nb(i) * diaul(i)$
$nbtw = nbtw + nb(i)$
1
$\rho_b = \rho_b / 2(dbe \times bbe)$
$\rho_w = \rho_w / 2(bweb \times dweb)$
dialb = dialb/nbtb
dialw = dialw/nbtw


Read reinforcement details in boundary element:
Diameter of transverse reinforcement (dias), spacing (s), number of bars in the c/s in the orthogonal direction (nco), number of bar in the c/s along the diogonal direction (ncd), number of peripheral longitudinal bars at top (nbart), at bottom (nbarb), on left side (nbarl), on right side (nbarr). (in nbarr and nbarl the top and bottom layer are also included)
Calculation of core area:
bbecore = bbe – 2*becov – dias
dbecore = dbe – 2*becov – dias
area = $\pi$ dias$^2$/ 4
sigarea = area*nco + area*ncd*sqrt(2)
rows = (dias$^2$ (bbecore+dbecore)/2.0)/ (bbecore+dbecore)
From subroutine 'subr3' calculate the ultimate axial load carrying capacity of the wall section ($P_o$)
If
$P_{axl} > P_o$
YES
Write "The axial load cannot be sustained by the section"
STOP
NO
Calculate maximum compressive train in concrete in:
web (wecmax) = 0.0035
boundary element (becmax) = 0.004 + 0.90*rows*($f_{yh}$ /300)

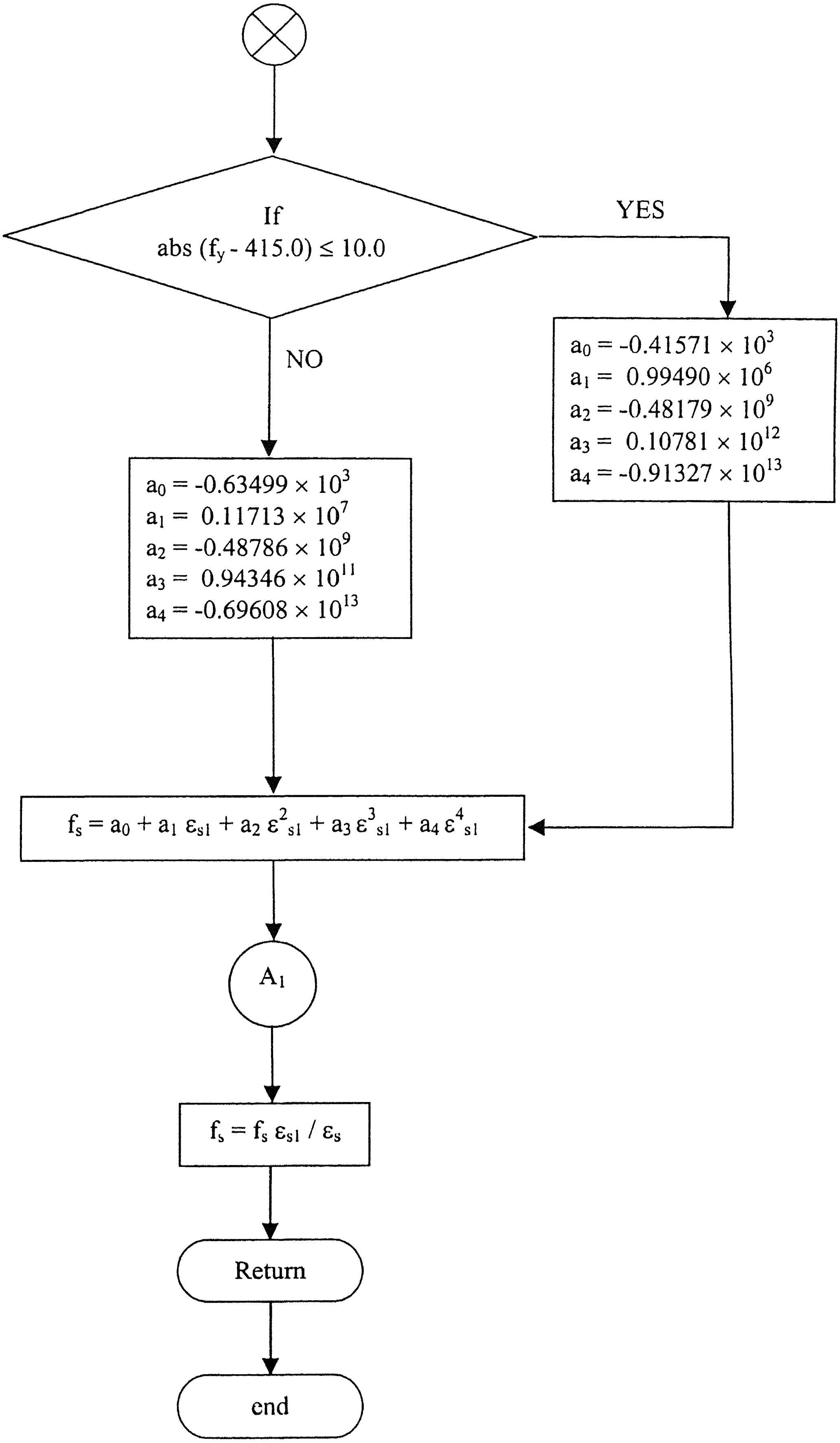

initialise, $\Delta\varepsilon_{c1} = \Delta\varepsilon_c$ , yield curvature ($\phi_y$) = 0.0, maximum moment ($M_{max}$) = 0.0, strain at the outer most concrete fibre ($\Delta\varepsilon_c$) = 0.0, number of iterations to depict complete moment-curvature curve (kk) = 0
From subroutine 'subr3' calculate the uniform strain in the cross-section due to applied axial load
B
Set, $\varepsilon_c = \varepsilon_c + \Delta\varepsilon_c$, $\phi = 0.0$, $\Delta\phi = 10^{-5}$
Number of iterations to calculate a single point on the moment-curvature curve (kn)=0
C
$\phi = \phi + \Delta\phi$, Set $P_{cal} = 0.0$
Check if the strain in the top most compression fiber (ecbbe) with the maximum allowed strain in confined concrete (becmax) and the strain in the extreme steel bar in the boundary element with the fracture strain of the longitudinal steel bar ($\varepsilon_{smax}$).
Divide the wall cross-section into n- laminae.
Depending on the $\varepsilon_c$ and $\phi$ calculate strain at the middle of the each lamina, calculate area of core concrete and cover concrete in each lamina.
Using subroutine 'subr1' calculate stress in each lamina. Compute compression force acting on the lamina and distance of its point of action from the centroid of the section.

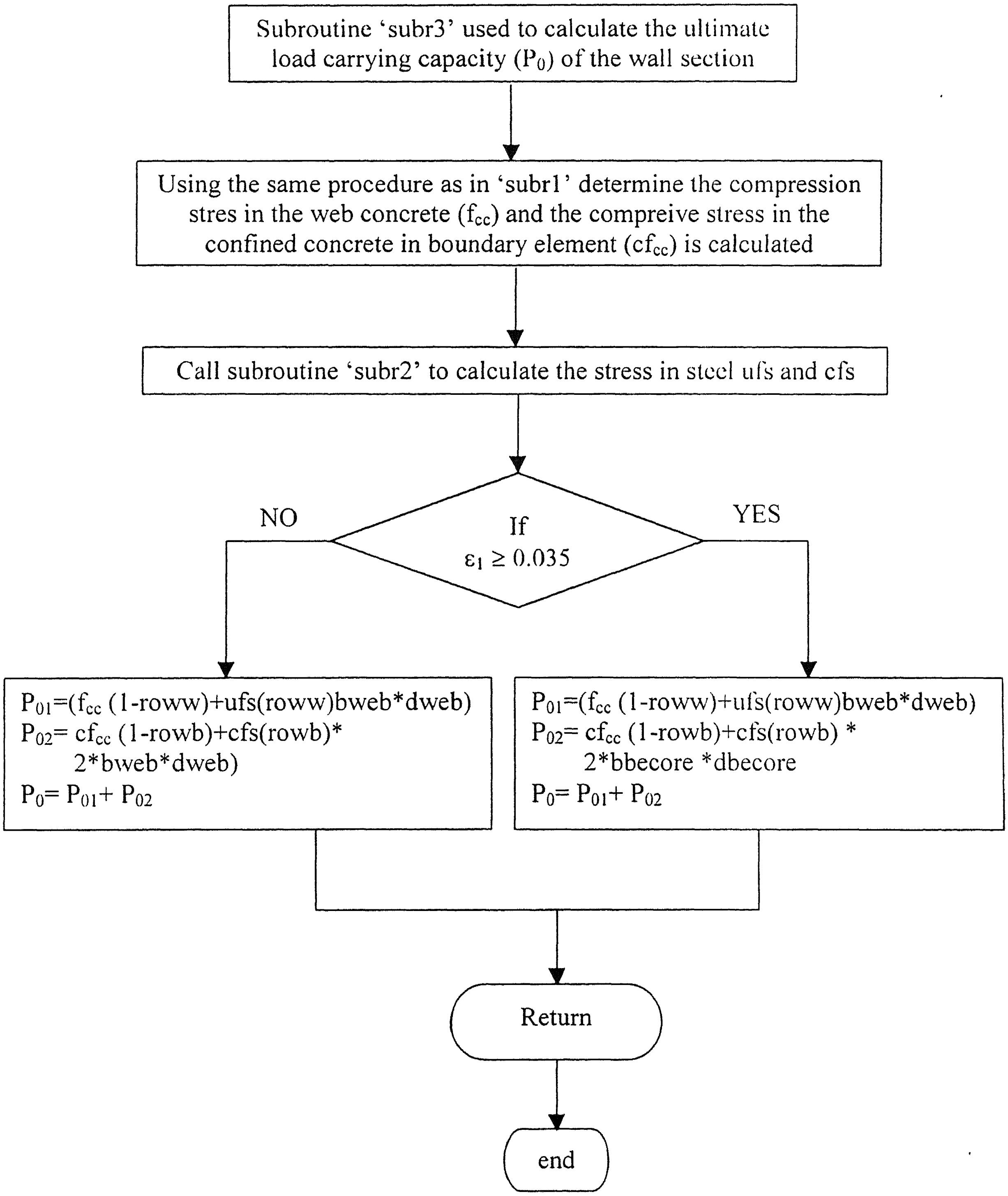

If
M ≤ 0.8 Mmax
YES
Write " The ultimate moment became less than 80% of the Maximum moment (Mmax)" and terminate the program.
Stop and End
NO
If
M ≥ Mmax
YES
Mmax = M
NO
Using εc and ϕ calculate the strain in the extreme steel bar (esmx) and the strain at the junction of the boundary element with the web (ecj)
If
esmx ≥ εsmax
YES
Write "bottom most steel bar reached fracture strain
Stop and End
NO
If
ecj ≥ 0.002
YES
Write "web concrete reached yield strain".
Stop and End
NO
B


Subroutine 'subr1' used to find stress in concrete ($f_c$) for a particular strain ($\varepsilon_c$)
Initialise $f_c = 0.0$
If $\varepsilon_c \leq 0.0$
YES
Return
NO
Calculate the equivalent uniform lateral pressure ($f_{le}$) due to confinement
$K_1 = 6.7\ f_{le}^{-0.17}$
Peak stress $f_{cc} = f_{ck} + k_1\ f_{le}$
$K = K_1 + f_{le}/f_{ck}$
Peak strain $\varepsilon_1 = 0.002\ (1+5k)$
Strain correponding to 85% of peak stress
$\varepsilon_{85} = 260\rho\varepsilon_1+0.0038$
If $\varepsilon_c \leq \varepsilon_1$
YES
$\eta = \varepsilon_c/\varepsilon_1$
$f_c = f_{cc}\ (2\eta-\eta^2)^{(1+2k)}$
$f_c = \min\ (f_c\ ,\ f_{cc})$
NO
$f_c = f_{cc} - 0.15\ f_{cc}\ (\varepsilon_c-\varepsilon_1)/(\ \varepsilon_{85}-\varepsilon_1)$
$f_c = \max\ (f_c\ ,\ 0.20f_{cc})$
Return
End



Subroutine 'subr2' used to find steel stress ($f_s$) for a particular strain ($\varepsilon_s$)
Initialise $f_s = 0.0$
If $\varepsilon_{s1} = abs(\varepsilon_s)$
YES
Return
NO
If $abs(f_y - 250.0) \leq 10.0$
YES
$C1 = \varepsilon_{s1} E_s$
$f_s = min(C1, f_y) \varepsilon_{s1}/\varepsilon_s$
Return
NO
$\varepsilon_1 = 0.80 f_y/E_c$
$\varepsilon_2 = f_y/E_c + 0.002$
If $\varepsilon_{s1} \leq \varepsilon_1$
YES
$f_s = \varepsilon_{s1} E_s$
NO
If $\varepsilon_{s1} \geq \varepsilon_2$
YES
$f_s = f_y$
$A_1$
NO


If
abs ($f_y$ - 415.0) ≤ 10.0
YES
NO
$a_0 = -0.41571 \times 10^3$
$a_1 = 0.99490 \times 10^6$
$a_2 = -0.48179 \times 10^9$
$a_3 = 0.10781 \times 10^{12}$
$a_4 = -0.91327 \times 10^{13}$
$a_0 = -0.63499 \times 10^3$
$a_1 = 0.11713 \times 10^7$
$a_2 = -0.48786 \times 10^9$
$a_3 = 0.94346 \times 10^{11}$
$a_4 = -0.69608 \times 10^{13}$
$f_s = a_0 + a_1\,\varepsilon_{s1} + a_2\,\varepsilon^2_{s1} + a_3\,\varepsilon^3_{s1} + a_4\,\varepsilon^4_{s1}$
$A_1$
$f_s = f_s\,\varepsilon_{s1} / \varepsilon_s$
Return
end


Subroutine 'subr3' used to calculate the ultimate load carrying capacity ($P_0$) of the wall section
Using the same procedure as in 'subr1' determine the compression stres in the web concrete ($f_{cc}$) and the compreive stress in the confined concrete in boundary element ($cf_{cc}$) is calculated
Call subroutine 'subr2' to calculate the stress in steel ufs and cfs
If $\varepsilon_1 \geq 0.035$
NO
YES
$P_{01}$=($f_{cc}$ (1-roww)+ufs(roww)bweb*dweb)
$P_{02}$= $cf_{cc}$ (1-rowb)+cfs(rowb)* 2*bweb*dweb)
$P_0 = P_{01} + P_{02}$
$P_{01}$=($f_{cc}$ (1-roww)+ufs(roww)bweb*dweb)
$P_{02}$= $cf_{cc}$ (1-rowb)+cfs(rowb) * 2*bbecore *dbecore
$P_0 = P_{01} + P_{02}$
Return
end

# Program Listing

```
C   CURVATURE DUCTILITY OF SHEAR WALLS WITH CONFINED BOUNDARY
C   ELEMENTS

          parameter (n=500)
          double precision es,ec,epu,delc,c,ee,ee1,ee2,e1,phimax
          dimension d(n),fct(n),fst(n),dst(n)
          dimension diaul(50),nb(50)
          common/block1/bweb,dweb,bbe,dbe,dtot,becov,wcov,dias,fyh,fck,s
          common/block2/bbecore,dbecore,nbart,nbarb,nbarl,nbarr,area,row,roww,rowb
          common/block3/iopt
          common/block4/fy,ye
          common/block5/e1,fcc
          common/block6/dialb

C   GEOMETRIC PROPERTIES OF SHEAR WALL WITH BOUNDARY ELEMENTS

          open(22,file='mphi')
          pi=3.1415927
          read(5,*) bweb, dweb, bbe, dbe, dtot, becov, wcov

C   PROPERTIES OF MATERIALS

          read(5,*) fck, fy, fyh, ye, esmax,
          read(5,*) delc, paxl nh
          write(6,105)
          write(6,205) bweb, dweb, bbe, dbe, dtot, becov, wcov, fck, fy, fyh, ye, esmax, nh, paxl

C   REINFORCEMENT DETAILS IN THE SHEARWALL

          rowb = 0.0
          roww = 0.0
          dialb = 0.0
          dialw = 0.0
          nbtb = 0.0
          nbtw = 0.0
          do i=1,nh
           read(5,*) dst(i), diaul(i), nb(i)
          enddo
          do i=1,nh
           if (dst(i).le.dbe.or.dst(i).ge.(dbe+dweb))then
             rowb=rowb+(nb(i)*pi*(diaul(i)/2.0)**2)
             dialb=dialb+nb(i)*diaul(i)
             nbtb=nbtb+nb(i)
           else
             roww=roww+(nb(i)*pi*(diaul(i)/2.0)**2)
             dialw=dialw+nb(i)*diaul(i)
```

```
       nbtw=nbtw+nb(i)
      endif
     end do
     rowb=rowb/(2*(bbc*dbc))
     dialb=dialb/nbtb
     roww=roww/(bweb*dweb)
     dialw=dialw/nbtw
     write(6,255) (dst(i), diaul(i), nb(i),i=1,nh)

C  REINFORCEMENT DETAILS IN BOUNDARY ELEMENT

     read(5,*) dias, s, nco,ncd
     read(5,*) nbart,nbarb,nbarl,nbarr
     bbecore=bbe-2.0*becov-dias
     dbecore=dbe-2.0*becov-dias
     area=pi*(dias/2.0)**2
     sigarea=area*nco+area*ncd*sqrt(2.0)
     row=sigarea/(s*(bbecore+dbecore))
     rows=(pi*dias**2*(bbecore+dbecore)/2.0)/(bbecore*dbecore*s)
     write(6,275) dias, s, nbart, nbarb, nbarl, nbarr

C  CALCULATION OF AXIAL LOAD CARRYING CAPACITY OF SHEARWALL

     write(6,305)
     epu=0.002
     call subr3(epu,p0)
     write(6,335) p0
C
     if(paxl.ge.p0) then
      write(6,345)
      goto 500
     endif
     wecmax = 0.0035
     becmax = 0.004+0.90*rows*fyh/300.0

C  INITIALISATION

     phiy=0.0
     umy=0.0
     phiu=0.0
     umu=0.0
     is=1
     rr=0.0
     pcal=0.0
     delc1=delc
     umax=0.0
     ec=0.0
     kk=0
     phimax=0.0
     delc=delc1
```

```
C  CALCULATION OF UNIFORM STRAIN ACROSS THE CROSS SECTION
C  UNDER THE APPLIED AXIAL LOAD

 210     ec=ec+delc
         call subr3(ec,pcal)
         if(pcal.le.paxl) goto 210
         ec=ec-delc
 220     ec=ec+delc
 250     kn=0
 230     delphi=0.00001
 240     phi=phi+delphi

C  CHECKING THE STRAIN IN THE TOP MOST COMPRESSION FIBRE WITH
C  THE MAXIMUM ALLOWED STRAIN IN CONFINED CONCRETE (becmax)
C  AND THE STRAIN IN THE EXTREME BAR IN THE BOTTOM BOUNDARY
C  ELEMENT WITH THE FRACTURE STRAIN IN THE LONGITUDIANAL STEEL

         esbbe=ec-phi*(dtot-becov-dias-dialb/2.0)
         ecbbe=ec-phi*(dtot-becov-dias/2.0)
         if(ec.ge.becmax.or.ecbbe.ge.becmax)then
         write(6,350)
         goto 499
         endif
         if(abs(esbbe).ge.esmax)then
         write(*,351)
         goto 499
         endif
         c=ec/phi
         if(c.gt.1.0e+10) then
           ec=ec+delc1
           goto 250
         endif
         pcal=0.0

C  CALCULATION OF FORCES IN CONCRETE:
C  THE SECTION IS DEVIDED INTO NUMBER OF LAMINAE.THE STRAIN AT
C  THE CENRE OF EACH LAMINA IS FOUND FROM THE DEPTH OF THE
C  LAMINA FROM TOP OF THE SECTION AND THE CURVATURE ASSUMED.
C  IF THE STRAIN AT THE TOP OF THE SECTION IS 'ec', THE DEPTH OF THE
C  ELEMENT FROM TOP IS 'd' AND 'phi' IS THE ASSUMED CURVATURE
C  THAN STRAIN AT THE CENTRE OF THE LAMINA IS ec-phi*d. THE
C  STRESS CORRESPONDING TO THIS STRAIN IS READFROM THE STRESS-
C  STRAIN RELATIONSHIP (CONFINED OR UNCONFINED, ACCORDING TO
C  THE DEPTH OF THE LAMINA). THE AREA OF EACH LAMINA IS
C  CALCULATED. AND THE FORCE IN EACH LAMINA IS GOT BY
C  (STRESS*AREA).

         do 270 i=1,n
         te=dtot/n
         d(i)=(i-0.5)*te
```

```
      h=d(i)
      ee=ec-phi*h

C  NEGLECTING THE CONTRIBUTION OF CONCRETE IN TENSION

      if(ee.le.0.0) then
       fct(i)=0.0
       goto 270
      endif
      call subr1(h,ee,fc)
      if((h.le.dbe).or.(h.ge.(dbe+dweb))) then
      ds=0.5*dtot-becov-0.5*dias
      de1=abs(dtot/2.0-h)+te/2.0
      de2=abs(dtot/2.0-h)-te/2.0
      ee1=ec-phi*(h-te/2.0)
      ee2=ec-phi*(h+te/2.0)
            if(de2.ge.ds) then
             acore=0.0
            else
             if(de1.le.ds) then
              acore=(bbe-2.0*becov-dias)*te
             else
              acore=(bbe-2.0*becov-dias)*(ds-de2)
             endif
            endif
C
            if(ee2.ge.becmax) then
             acon=acore
            else
             if(ee1.gt.becmax) then
              acon=acore+(2.0*becov-dias)*te*((becmax-ee2)/(ee1-ee2))
             else
              acon=bbe*te
             endif
            endif
            goto 260
      else

C  NEGLECTING THE FORCE IN THE WEB CONCRETE WHOSE STRAIN IS
C  MORE THAN 0.0035

            if(ee.le.0.0035)then
             acon=bweb*te
            else
             acon=0.0
            endif
      endif
260   fct(i)=fc*acon
      pcal=pcal+fct(i)
270   continue
```

```
C   CALCULATION OF FORCES IN REINFORCEMENT

      do ii=1,nh
       dii=dst(ii)
       es=ec-phi*dst(ii)
       if(es.le.0.0)then
         fc=0.0
         goto 280
       endif
       call subr1(dii,es,fc)
 280   call subr2(es,fs)
       fst(ii)=(fs-fc)*nb(ii)*pi*(diaul(ii)/2.0)**2
       pcal=pcal+fst(ii)
      enddo
C
      kn=kn+1
      if(kn.gt.200) then
       goto 220
      endif
C
      if((abs(pcal-paxl)/paxl).gt.0.001) then
       if(pcal.le.paxl) then
         phi=phi-delphi
         delphi=delphi/10.0
       endif
       goto 240
      else

C   CALCULATION OF MOMENT ABOUT CENTROIDAL AXIS

       um=0.0
       do i=1,n
         h=dtot/2.0-d(i)
         um=um+fct(i)*h
       enddo
       do ii=1,nh
         hb=dtot/2.0-dst(ii)
         um=um+fst(ii)*hb
       enddo
C
       kk=kk+1
       if(kk .eq. 1) write(6,315)
C
       write(6,325)phi, um, ec, esmx
       um=um/1000000.0
       if(um.le.(0.8*umax))then
         write(*,349)
         goto 499
       endif
       if(um.ge.umax)umax=um
```

```
        esmx=ec-phi*(dtot-becov-dias-(dialb/2.0))
        ecj=ec-phi*dbe
       ey=fy/yc
       if((abs(esmx).ge.ey)) then
         write(*,*)'bottom most steel bar reached yield strain'
       endif
       if(ecj.ge.0.002)then
         write(*,*)'web concrete reached yield strain'
       endif
       phiy=phi
       umy=um
      write(22,*)phi,um
      umu=um
      phiu=phi
      goto 220
     endif
     if(kk.gt.500)then
     write(6,355)kk
     endif
499  write(*,*)'ultimate moment = ',umu,' kN-m'
     write(*,*)'ultimate curvature = ',phiu
     write(*,*)'Maximum moment = ',umax,' kN-m'
     goto 500

C  FORMATTING

105  format(2x,'*********************************************************
     *****',/,
   * 2x,'*',55x,2x,'*',/,
   * 2x,'*',2x,'ANALYSIS OF SHEARWALL WITH CONFINED BOUNDARY
ELEMENTS
   *',2x,'*',/,
   * 2x,'*',57x,'*',/,
   * 2x,'*********************************************************
   *',///)
115  format()
125  format(/)
135  format(//)
205  format(4x,'INPUT DATA',/,
      * 4x,'GEOMETRIC PROPERTIES OF SHEAR WALL',/,
      * 4x,'***********************************',/,
   * 4x,'BREADTH OF WEB                    = ',f7.2,' mm',/,
   * 4x,'DEPTH OF  WEB                     = ',f7.2,' mm',/,
      * 4x,'BREADTH OF BOUNDARY ELEMENT        = ',f7.2,' mm',/,
   * 4x,'DEPTH OF BOUNDARY ELEMENT         = ',f7.2,' mm',/,
   * 4x,'TOTAL DEPTH OF SHEAR WALL         = ',f7.2,' mm',/,
   * 4x,'COVER IN BOUNDARY ELEMENT         = ',f7.2,' mm',/,
   * 4x,'COVER IN WEB                      = ',f7.2,' mm',/,
   * 4x,'CHARACTERISTIC STRENGTH OF CONCRETE  = ', f8.1,' MPa',/,
   * 4x,'YIELD STRENGTH OF LONGITUDINAL STEEL = ', f8.1,' MPa',/,
```

```
     * 4x,'YIELD STRENGTH OF TRANSVERSE STEEL    = ',f8.1,' MPa',/,
     * 4x,'YOUNGS MODULUS OF ELASTICITY          = ',f8.1,' MPa',/,
     * 4x,'FRACTURE STRAIN OF LONGITUDINAL STEEL = ',f8.4,/,
     * 4x,'NUMBER OF LAYERS OF REINFORCEMENT     = ',I3,/,
     * 4x,'AXIAL LOAD ON THE SECTION             = ', e12.5,' N',/)
C
215  format(2x,'* MOMENT-CURVATURE CURVE IS REQUIRED')
225  format(2x,'* VALUE OF DUCTILITY IS REQUIRED')

255  format(4x,'REINFORCEMENT DETAILS IN SHEARWALL',/,
     *      4x,'*********************************'
     *,/,4x,'DISTANCE FROM TOP',4x,'DIAMETER (in mm)',4x,'NO. OF BARS'
     *,/,(8x,f7.2,12x,f5.2,13x,i3))

275   format(/,4x,'REINFORCEMENT DETAILS IN BOUNDARY ELEMENT',/,
     *       4x,'*****************************************',/,
     *4x,'DIA. OF TRANSVERSE BARS IN BOUNDARY ELEMENTS= ',f6.2,' mm',/,
     *4x,'SPACING OF TRANSVERSE REINFORCEMENT = ',f6.2,' mm',/,
     *4x,'NO. OF LONGITUDINAL BARS AT TOP OF THE SECTION    =',i2,/,
     *4x,'NO. OF LONGITUDINAL BARS AT BOTTOM OF THE SECTION =',i2,/,
     *4x,'NO. OF LONGITUDINAL BARS AT LEFT OF THE SECTION   =',i2,/,
     *4x,'NO. OF LONGITUDINAL BARS AT RIGHT OF THE SECTION  =',i2,/)
305  format(1x,'**********OUTPUT********',//)
315  format(4x,'CURVATURE',8x,'MOMENT',8x,'MAX. COMP.',8x,'MAX. TEN.'
     * ,/,20x,'(in N-mm)',4x,'STRAIN IN CONC.',2x,'STRAIN IN STEEL',//)
325  format(1x,e12.5,4x,e12.5,4x,e12.5,3x,e12.5)
335  format(4x,'ULTIMATE AXIAL LOAD = ',e12.5,' N',/)
345  format(1x,'****  THE AXIAL LOAD CANNOT BE SUSTAINED  ****',/,
     * 2x,'***       BY THE SECTION            ***')
349  format(4x,'THE ULTIMATE MOMENT BECAME LESS THAN 80% OF THE
     * MAXIMUM MOMENT',/,6X,'AND THE PROGRAM TERMINATED')
350  format(4x,'CONCRETE IN THE TOP BOUNDARY ELEMENT REACHED
     * ULTIMATE STRAIN',6X,'AND THE PROGRAM TERMINATED')
351  format(4x,'EXTREME STEEL BAR IN BOTTOM BOUNDARY ELEMENT
     * REACHED FRACTURE STRAIN',/,6X,'AND THE PROGRAM TERMINATED')
355  format(4x,'THE SOLUTION DIDNOT CONVERGE WITHIN',2X,i3, 1X,
     * 'ITERATIONS')
500   stop
      end


C     CONCRETE STRESS-STRAIN MODEL
C     1. UNCONFINED SECTION: IS:456 (1978)
C     2. CONFINED SECTION: SAATCIOGLU, M., AND RAZVI, S. R. C
      ASCE,118(6),JUNE,1992, PP.1590-1607.

      subroutine subr1(h,ec,fc)
      double precision ec,e1
      common/block1/bweb,dweb,bbe,dbe,dtot,becov,wcov,dias,fyh,fck,s
      common/block2/bbecore,dbecore,nbart,nbarb,nbarl,nbarr,area,row,roww,rowb
```

```
      common/block3/iopt
      common/block4/fy,ye
      common/block5/e1,fcc
      common/block6/dialb
C
      fc=0.0
      if(ec.le.0.0) goto 111
C
      if((h.le.dbe).or.(h.ge.(dbe+dweb))) then
       fl1=2.0*area*fyh/(s*bbecore)
       ak21=.26*sqrt(bbecore**2/(s*((bbecore-dias-dialb)/(nbart-1))*fl1))
       ak21=min(ak21,1.0)
       fle1=ak21*fl1
       fl2=2.0*area*fyh/(s*bbecore)
       ak22=.26*sqrt(bbecore**2/(s*((bbecore-dias-dialb)/(nbarb-1))*fl2))
       ak22=min(ak22,1.0)
       fle2=ak22*fl2
       fl3=2.0*area*fyh/(s*dbecore)
       ak23=.26*sqrt(dbecore**2/(s*((dbecore-dias-dialb)/(nbarl-1))*fl3))
       ak23=min(ak23,1.0)
       fle3=ak23*fl3
       fl4=2.0*area*fyh/(s*dbecore)
       ak24=.26*sqrt(dbecore**2/(s*((dbecore-dias-dialb)/(nbarr-1))*fl4))
       ak24=min(ak24,1.0)
       fle4=ak24*fl4
       fle=((fle1+fle2)*bbecore+(fle3+fle4)*dbecore)/(2.0*(bbecore+dbecore))
       ak1=6.7*fle**(-0.17)
       fcc=fck+ak1*fle
       ak=ak1*fle/fck
       e1=0.002*(1.0+5.0*ak)
       e85=260.0*row*e1+0.0038
       if(ec.le.e1) then
        eta=ec/e1
        fc=fcc*(2.0*eta-eta**2)**(1.0/(1.+2.0*ak))
        fc=min(fc,fcc)
       else
        fc=fcc-0.15*fcc*(ec-e1)/(e85-e1)
        cfc=max(fc,(0.20*fcc))
       endif
       goto 111
      else
       e1=0.002
       fcc=fck
       if(ec.le.e1) then
        eta=ec/e1
        fc=fcc*(2.0*eta-eta**2)
        fc=min(fc,fcc)
       else
        fc=fcc
      endif
```

```
      endif
 111  return
      end

C     STEEL STRESS-STRAIN MODEL
C     1. MILD STEEL BARS (FE 250)
C     2. HYSD BARS (FE 415 AND FE 500) (AS PER IS:456, 1978)

      subroutine subr2(es,fs)
      double precision es
      common/block4/fy,ye
      fs=0.0
      esc=abs(es)
      if(esc.le.0.0) return
      if(abs(fy-250.0).le.10.0) then
       c1=esc*ye
       fs=min(c1,fy)*esc/es
       return
      endif
      e1=0.80*fy/ye
      e2=0.002+fy/ye
      if(esc .le. e1) then
        fs=esc*ye
      else
        if(esc .ge. e2) then
          fs=fy
        else
          if(abs(fy-415.0) .le. 10.0) then
            a0=-0.41571e+03
            a1=0.99490e+06
            a2=-0.48179e+09
            a3=0.10781e+12
            a4=-0.91327e+13
          else
                a0=-0.63449e+03
                a1=0.11713e+07
                a2=-0.48786e+09
                a3=0.94346e+11
                a4=-0.69608e+13
          endif
          fs=a0+a1*esc+a2*esc**2+a3*esc**3+a4*esc**4
        endif
      endif
    fs=fs*esc/es
      return
      end

C  SUBROUTINE FOR CALCULATION OF AXIAL LOAD CARRYING
C  CAPACITY OF SHEARWALL
```

```
      subroutine subr3(ee,p0)
      double precision ee,e1
      common/block1/bweb,dweb,bbc,dbc,dtot,bccov,wcov,dias,fyh,fck,s
      common/block2/bbecore,dbecore,nbart,nbarb,nbarl,nbarr,area,row,roww,rowb
      common/block3/iopt
      common/block4/fy,ye
      common/block5/e1,fcc
      common/block6/dialb
C
      p0=0.0
      if(ee.le.0.0) return
      fcc=fck
      if(ee.le.0.002)then
       eta=ee/0.002
       fcu=fcc*(2.0*eta-eta**2)
       ufc=min(fcu,fcc)
       else
       ufc=fcc
      endif
C
      fl1=2.0*area*fyh/(s*bbecore)
      ak21=0.26*sqrt(bbecore**2/(s*((bbecore-dias-dialb)/(nbart-1))*fl1))
      ak21=min(ak21,1.0)
      fle1=ak21*fl1
      fl2=2.0*area*fyh/(s*bbecore)
      ak22=0.26*sqrt(bbecore**2/(s*((bbecore-dias-dialb)/(nbarb-1))*fl2))
      ak22=min(ak22,1.0)
      fle2=ak22*fl2
      fl3=2.0*area*fyh/(s*dbecore)
      ak23=0.26*sqrt(dbecore**2/(s*((dbecore-dias-dialb)/(nbarl-1))*fl3))
      ak23=min(ak23,1.0)
      fle3=ak23*fl3
      fl4=2.0*area*fyh/(s*dbecore)
      ak24=0.26*sqrt(dbecore**2/(s*((dbecore-dias-dialb)/(nbarr-1))*fl4))
      ak24=min(ak24,1.0)
      fle4=ak24*fl4
      fle=((fle1+fle2)*bbecore+(fle3+fle4)*dbecore)/(2.0*(bbecore+dbecore))
      ak1=6.7*fle**(-0.17)
      cfcc=fck+ak1*fle
      ak=ak1*fle/fck
      e1=0.002*(1.0+5.0*ak)
      e85=260.0*row*e1+0.0038
      if(ee.le.e1) then
       eta=ee/e1
       fc=cfcc*(2.0*eta-eta**2)**(1.0/(1.0+2.0*ak))
       cfc=min(fc,cfcc)
      else
       fc=cfcc-0.15*cfcc*(ee-e1)/(e85-e1)
       cfc=max(fc,(0.20*cfcc))
      endif
```

```
C
      call subr2(ee,ufs)
      call subr2(e1,cfs)
      if(e1.ge.0.0035) then
        p01=(fcc*(1-roww)+ufs*roww)*bweb*dweb
        p02=(cfcc*(1-rowb)+cfs*rowb)*bbecore*dbecore*2
        p0=p01+p02
      else
        p01=(fcc*(1-roww)+ufs*roww)*bweb*dweb
        p02=(cfcc*(1-rowb)+cfs*rowb)*bbe*dbe*2
        p0=p01+p02
      endif
      return
      end
```